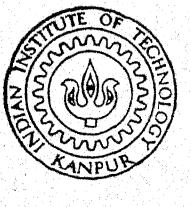
BEHAVIOUR OF REINFORCED AND PRESTRESSED CONCRETE FOLDED-PLATES WITH TIE BARS

By

P. INDUBHUSHAN



DEPARTMENT OF CIVIL ENGINEERING

INDIAN INSTITUTE OF TECHNOLOGY. KANPUR

BEHAVIOUR OF REINFORCED AND PRESTRESSED CONCRETE FOLDED PLATES WITH TIE BARS

Thesis submitted in partial fulfilment
of the requirements for the
degree of

MASTER OF TECHNOLOGY

in

CIVIL ENGINEERING

by

PATTAIKUNI INDIRAGHAN

CE-1969-M-IND-BEH

DEPARTMENT OF CIVIL ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY, KANPUR, U.P.(India)

JANUARY - 1969

The thesis on "Behaviour of Reinforced and Prestressed Concrete Folded Plates With Tie Bars" by Mr. P. Indubhushan, as a partial fulfilment towards the degree of M. Tech. in Civil Engineering is accepted.

KANPUR

January, 1969.

Dayaratnam

(P. Dayaratnam)
Associate Professor
and Chairman of
Thesis Committee

ACKNOWLEDGEMENT

The writer expresses his deep sense of gratitude to Dr. P. Dayaratnam for his able and efficient guidance, the patience shown and the encouragement given through out this research project work.

The writer is thankful to Mr. S.C. Goel for the help rendered by him during this project work and the Technical staff of the Structural Engineering laboratory of the Institute for the help during this project work.

C O N T E N T S

		Page No.
	Notations	
1.	INTRODUCTION	1
	1.1 Review of Literature on Associated Subject	1
	1.2 Proposed Investigation ...	5
2.	EXPERIMENTAL WORK	7
	2.1 Specimen	7
	2.2 Materials	9
	2.2.1 Concrete ...	9
	2.2.2 Steel	9
	2.3 Experimental Set-up ...	9
	2.3.1 Loading ...	9
	2.3.2 Instrumentation ...	10
	2.4 Testing	12

		Page No.
3.	RESULTS AND DISCUSSION	15
	3.1 R.C.C. Folded Plates	15
	3.2 Prestressed Concrete Folded Plates	16
	3.3 Effect of Prestressing the Ties	19
	3.4 Ultimate Strength	38
	3.4.1 R.C.C. Folded Plates ...	38
	3.4.2 Prestressed Concrete Folded Plates	39
	3.5 Secondary Stresses from Experimental Observations	40
	3.5.1 Secondary Stresses of Reinforced Concrete Folded Plates.	41
	3.5.2 Secondary Stresses of Prestressed Concrete Folded Plates.	43
	3.6 Cracking Pattern and Modes of Failure	45
4.	CONCLUSIONS	51
	REFERENCES	55
	APPENDIX-ULTIMATE STRENGTH CALCULATIONS	57
	PHOTOGRAPHS	63

NOTATIONS

A = cross sectional area

A_s = area of steel

a = depth of rectangular stress block

b = equivalent rectangle width

d = equivalent depth of the centroid of tension reinforcement from top fibre.

E = Young's modulus of concrete

FP0 = prestressed concrete folded plate without any tie

FP1 = prestressed concrete folded plate with one tie

FP2 = prestressed concrete folded plate with two ties

FP3 = prestressed concrete folded plate with three ties

FP4 = prestressed concrete folded plate tested with one, two and three ties.

FR0 = reinforced concrete folded plate without any tie

FR1 = reinforced concrete folded plate with one tie

FR2 = reinforced concrete folded plate with two ties

f'_c = 4" cube compressive strength of concrete

f_{su} = ultimate tensile strength of reinforcement steel

f_y = yield strength of reinforcement steel

h = finite difference sub-interval

I = moment of inertia

L = span

M_u = ultimate moment

p = tension steel ratio

p' = compression steel ratio

P = prestressing force

P_u = total ultimate load

P_{su} = ultimate load taken by steel

t = plate thickness of the folded plate

v = deflection along the plate

w = deflection perpendicular to plate

ϕ = diameter of bar

σ = stress

1. INTRODUCTION

1.1 Review of literature on associated subject:

A brief review on the ultimate capacity of reinforced concrete folded plates and the effect of tie bars or diaphragms on the behaviour and strength is presented in this section. The object of the present investigation is to study the effect of the spacing of tie bars on the stiffness and ultimate capacity of reinforced and prestressed concrete folded plates. Hence the review of literature is primarily restricted to the object of the investigation.

Glanville[1*] tested three V-shaped full-scale pretensioned folded plate roofs with lateral tie bars between the peaks at mid span, quarter points, and the supports. After applying a test load producing theoritical ultimate moment at mid-span, the load was released and then the lateral tie bars were removed. He observed that during initial stages of loading well distributed series of vertical flexural cracks in the maximum moment section were developed and the mid-span tie bars developed tension rapidly as the theoritical ultimate load was reached, indicating the tendency of the unit to flatten out. After

* Super script indicates the reference number.

removing the tie bars the specimens were reloaded to collapse which occurred suddenly by a flattening out of the unit at a test load at which it would appear that flexural collapse was also imminent. It was shown from his observations that the ridge deflection of folded plates with and without tie bars are similar.

Scheighofer and Seethaler[2] have tested a 7ft. wide and 46.67 ft. long precast and prestressed folded plate of Y-shape to determine the economical spacing of ties. As an accurate theoritical analysis of complicated shape of folded plate with ties would be exceedingly cumbersome, they have used the data of a plastic model using a scale of 1/16. First they have used four ties, one at each end and the remaining two at the third points. They have recorded the strains in the ties by successively placing a unit load on points following the longitudinal center-line of the unit. The force versus position influence diagrams for the applied unit load were drawn and from the ordinates of both tie influence curve (ties on one side of the centerline of the unit), they found that the tie spacing was indeterminate because most of the unit load placed was transmitted to the stem by transverse bending and only about 1/20 of the load was carried by the neighboring ties. The addition of three ties with a new tie spacing of 1.33 times the width of the model caused

86 percent of the unit load, when applied midway between two ties, to be absorbed by the ties and the remainder by slab action. So they adopted this tie spacing for the final design of the prototype which proved highly economical. Therefore they recommended a tie spacing of 1.33 times the width of the unit as it results in extremely thin flanges. They observed that the unit behaved like a beam in the longitudinal direction.

Ali[3] has studied both analytically and experimentally folded plates with intermediate stiffners. He used Aluminum models for his experimental investigation. He has shown that with sufficiently close spacings of stiffners the longitudinal stresses approach a straight line distribution. The application and the limitations of the beam method for folded plates with intermediate stiffeners was discussed by Winter and Pei[4]. The effect of transverse stiffners in closed prismatic structures was discussed by Cramer[5]. He observes that the deformation of the stiffener, which may be a wall, floor, rib, or the like, is negligible compared with the displacement of the slab. He further observes that a large reduction of stresses and the almost linear variation of stress results from the provision of the floor. Approximate methods of spacing and proportioning of the intermediate

diaphragms were discussed by Huang[6]. He observes that the spacing of intermediate diaphragms depend upon the material, dimensions and shape of the roof structure. He further observes that the thickness, depths and the degree of the connecting angles between the individual plates, and the rigidity of the intermediate diaphragms are important factors in deciding the spacing of the intermediate diaphragms. He concluded that the general criterion could be expressed by the ratio y_s/y_b where y_s is the deflection of a plate of span L_s due to the secondary effect and y_b is the assumed theoritical deflection of a plate of a hinged plate roof with intermediate diaphragms. He also concluded that by the insertion of intermediate diaphragms the whole cross section remains without distortion under the action of external loads thereby permitting the application of ordinary beam theory.

Edwards[7] tested a V-shaped prestressed concrete folded plate, constructed of two 2½ in. flat slabs, 51 in. wide and 50 ft. long, joined at the ridge by welding and then grouting the joint. The folded plate unit was subjected to a uniformly distributed live load in increments upto 158 psf. No tie bar was provided.

The folded plate structure was designed for a safe live

load of 30 psf. using the simple beam theory. No attempt was made to describe the behaviour of the folded plate. Chacos and Scalzi[6] reported the test to collapse of three plate ⌒ shaped reinforced mortar model of 60 in. span, 5.07 in. rise and width of 43.72 in. with a plate thickness of 0.5 in. The test structure was loaded with bricks on the top horizontal plate. The authors computed the ultimate collapse moment using the Whitney stress block and simple beam theory and was compared with test moment. The ratio was found to be 0.982.

Aldridge[3] reported tests of four folded plate models one of which was a prestressed model. He used a symmetrically placed uniformly applied monotonic load to failure. He concluded that the ultimate load carrying capacity of reasonably proportioned reinforced concrete folded plate structures may be predicted quite closely with ordinary ultimate strength procedures for flexural members.

1.2 Proposed Investigation:

Much work is not done to predict the behaviour of folded plate structure with ties. Ties or transverse diaphragms are supposed to add to the rigidity of the folded plate structure. It may be of help to have some test data of the behaviour of the folded plate structure

with transverse ties. It was proposed to study the effect of tie bars on the behaviour of both reinforced and prestressed concrete folded plates. To date, not much of test data has been published. Therefore it was proposed to study the behaviour of the folded plate for different tie bar spacings, the effect of the tie bars on the ultimate strength of the folded plate and the cracking pattern of the folded plate.

The scope of the experimental work proposed has been limited to the observation of load - deflection responses of reinforced and prestressed concrete folded plate structures with and without tie bars and the tie bar force-deflection responses of the folded plates with ties for no external load conditions. All the specimens were simply supported on diaphragms with two point load on the ridge at one third span. The primary variable for the experimental program was the spacing of the ties.

2. EXPERIMENTAL WORK

2.1 Specimen:

All the folded plates were of 154 cm. long (c/c of diaphragms) with a rise of 15 cms., All the specimens were of two plates with V-shape, the interior angle between the two plates being 120°. The total length of all the folded plates was 172 cms. including the overhang on either side. The thickness of the plates was 2.5 cms. The diaphragms were of 4 cms. thick. The dimensions of the folded plate are shown in Fig. 2.1.1.

The specimens were devided into two sets. The first set consists of three reinforced concrete specimens and the second consists of five prestressed concrete ones.

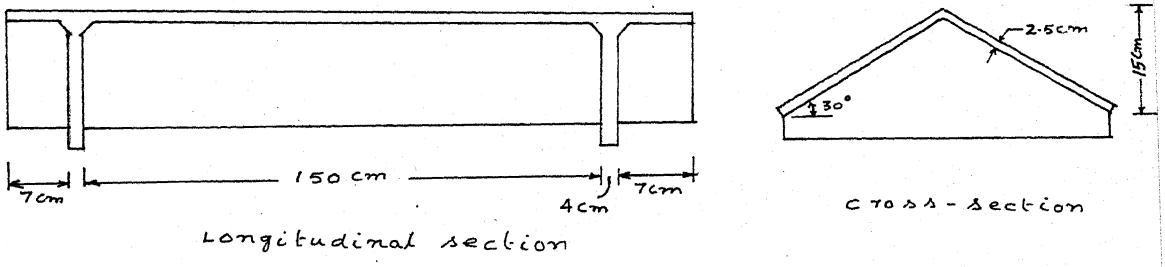


Fig. 2.1.1 Dimensions of Specimen

Out of the three reinforced folded plates the first one FRO * was provided with 9 bars of 1/8 in. Ø at 3 in. c/c. The transverse reinforcement consisted of 1/8 in. dia. bars at 3 in. c/c bent to V-shape with an

*The specimens are given notation as FRO, FPO etc. as indicated in the notations.

interior angle of 120°. In addition to the main longitudinal reinforcement 1/8 in. dia. bars of 60 cms. long were provided in the longitudinal direction at the ends in between main longitudinal reinforcement. The second and third specimens FR1 and FR2 respectively were provided with a longitudinal reinforcement of 7 bars of 1/8 in. dia at 3 in. c/c, the other reinforcement being same as FR0.

The ordinary reinforcement of all the prestressed folded plates were same as that of FR1 or FR2. Four 5 mm. dia. prestressing wires were provided for each specimen, two wires in each plate. The specimens were post-tensioned. The location of prestressing wires which were straight is shown in fig. 2.1.2.

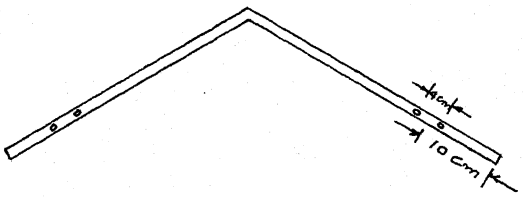


Fig. 2.1.2 Location of Prestressing Wires.

The tie bars were 7 mm. dia. bars of high tensile steel with threads at each end. They were secured to the folded plate by passing them through holes drilled in the folded plate and tightened through nuts.

2.2 Materials:

2.2.1 Concrete: Microconcrete was used for casting the specimens. A mix of 1:1:2 with a water cement ratio of 0.5 was used. The coarse aggregate used was 1/4 in. size.

2.2.2 Steel: Ordinary reinforcing steel: For longitudinal and transverse reinforcement 1/8 in. dia. bars with an ultimate strength of 7960 kg/cm^2 was used. High tensile steel: For prestressed folded plates 5 mm. dia. high tensile steel wires with an ultimate strength of 17625 kg/cm^2 were used.

2.3 Experimental Set Up:

The folded plates were tested on a self-straining loading frame. The folded plate was supported on cross-beams provided in the longitudinal direction. One diaphragm of the folded plate was supported on the channel section placed across the longitudinal cross beams. The other diaphragm was on roller support. The frame had longitudinal corss beams at the top to support the hydraulic jack.

2.3.1 Loading: A two point load was applied at the one third points of the span. This was achieved by placing two cast iron V blocks over the ridge points and an I-beam

over the V-blocks. The load was applied at the centre of this I-beam. The load was applied by a hydraulic jack at increments. Plaster of Paris packing was used in between the folded plate ridge and the V-block to avoid crushing of concrete due to direct contact of the V-block. The loading arrangement is shown in Fig. 2.3.1.

2.3.2 Instrumentation: Deflection dial gauges were used to measure the deflections of various points. Dial gauge locations are shown in Fig. 2.3.2.

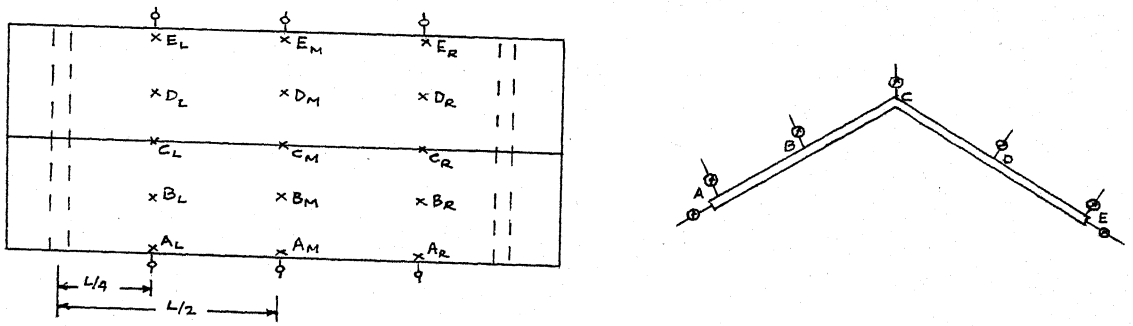


Fig. 2.3.2 Dial Gauge Locations.

Dial gauges at A, B, D and E were perpendicular to plate, the dial gauge at ridge i.e. C was vertical. Other six dial gauges provided at side were along the plate direction.

To measure the strain in the tie bars two SR-4 strain gauges on each tie bar were provided. The dial gauges were supported by an independent frame fabricated with slotted angles for this purpose. The frame deflections

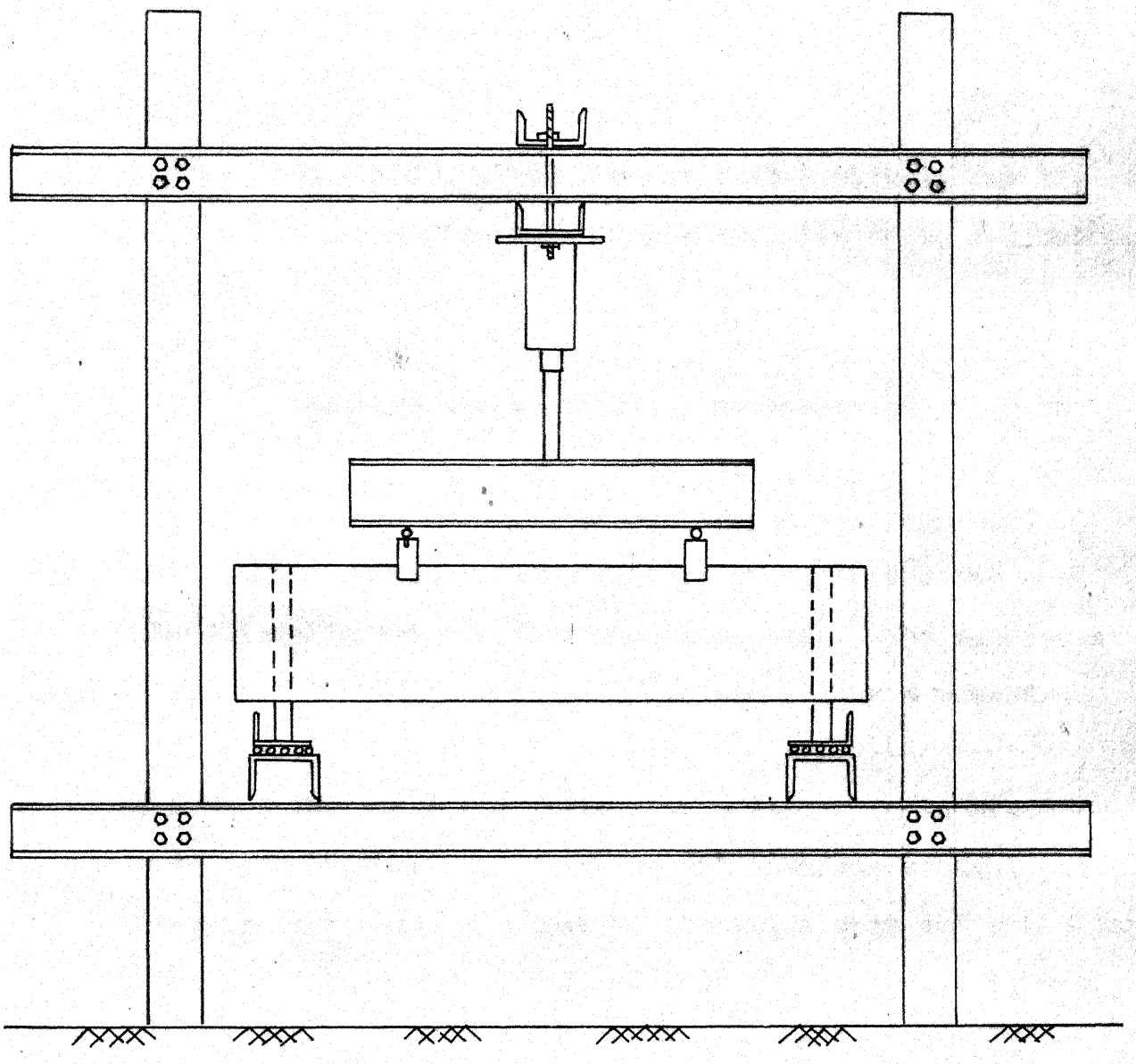


FIG.2.31 LOADING ARRANGEMENT

were measured by providing two dial gauges below both the folded plate support points.

2.4 Testing:

The folded plate FH0 was tested without any tie bar. FH1 was tested with one central tie bar. FH2 was tested with two tie bars at one third points. In all the cases the loading and dial gauge arrangement was same. The load was applied in increments and at each load level the dial gauge readings were taken. In the case of FH1 and FH2 the strain gauge readings were also taken. The cracking pattern was observed for each load level after the first crack started. The specimens were tested upto failure to find the ultimate moment capacity.

The testing of prestressed specimens consisted of: FP0 without any tie bar, FP1 with one central tie bar, FP2 with two tie bars at one third span and FP3 with three tie bars one at mid span and the other two at one third span. The prestressing force applied in all the cases was constant force of 8000 kg., i.e. 2000 kg. force was applied on each prestressing wire. The prestressing was done by post-tensioning the wires using Gifford Udall prestressing jack. For prestressed specimens also the loading and dial gauge arrangement was same.

For these models also the load was applied in increments and at each load level the dial gauge readings, and strain gauge readings were taken. The cracking pattern was observed. All the specimens were loaded to failure to find out the ultimate strength.

The effect of prestressing the tie bars was studied by prestressing the tie bars without any external load. In the case of FR1 the tie bar was not prestressed. It was a plain tie bar. The tie bars of FR2 were prestressed by turning the nuts provided for this purpose. The dial gauge readings and the strain gauge readings were taken at particular prestressing force on the tie bars. while prestressing the tie bars the folded plate was free of any external load.

In the case of prestressed folded plates the tie bars of FP1, FP2 and FP3 were prestressed without any external load on the folded plate. Here also the prestressing force on the tie bars was applied by turning the nuts provided for this purpose. At each level of prestressing force the dial gauge readings were taken. Careful observation was made to study any crack formation.

The folded plate FP4 was tested with one, two and three tie bars upto a load of 1820 kg. respectively to check the elastic behaviour of the folded plates tested. First it was loaded upto 1820 kg. in increments with three ties prestressed and the dial gauge readings

and the strain gauge readings were taken at each load level. Then it was unloaded and now it was provided with two tie bars. These tie bars were also prestressed and then it was loaded upto 1820 kg. in increments and the usual dial gauge and strain gauge readings were taken. At this stage it was unloaded. Now a single tie bar at the mid-span was provided. This tie also was prestressed and then it was loaded in increments. At every stage of loading the dial gauge and strain gauge readings were taken.

3. RESULTS AND DISCUSSION

3.1 R.C.C. Folded Plates: From the dial gauge observations for various load levels at different points, load-deflection curves were drawn. Load-deflection curves for various points can be seen in Figs. 3.1.1 to 3.1.8. The load-deflection curves clearly show the effect of tie bars. In general the deflections are reduced with the increasing number of ties. The sudden change in vertical deflections at ridge of mid-span C_M shows the efffect of cracking (Fig.3.1.1). During elastic range the deflection of C_M for FR1 is less than that for FR2. This shows the local effect of the tie. It is seen that during elastic range the deflection of C_M of FR1 and FR2 is much less than that for FR0 though the reinforcement in FR0 was more. But during inelastic range the deflection of FR0 is less. This is because of the extra reinforcement of 2 bars of 1/8 in. dia. in the longitudinal direction. From figure 3.1.2 and 3.1.3 it is seen that the deflection (perpendicular to plate) near the free edges and at the centre of the plate at mid span are less for folded plate with increasing number of ties. The local effect of tie is not much in this case. During elastic range the deflections of A_M (or B_M) and D_M(or D_M) of FR1 and FR2 are much less than that of FR0. In the inelastic range the deflections of FR0 is less because of

extra reinforcement. Here also the sudden change of curvature of the load-deflection curves shows the effect of cracking.

From figure 3.1.4 to 3.1.6 the load-deflection behaviour of points C, A (or S) and B(or D) at quarter span are seen. In these cases also the vertical deflection of C and the deflections perpendicular to plate of points A and B for folded plates with increasing number of ties are less. In the elastic range the deflections of FP2 are less than that of FP1 as may be expected as in the case of FP2 the ties were more closer to quarter span. Here also the effect of cracking on deflections is clearly seen. The load deflection v along the plate at quarter and mid spans are shown in figs 3.1.7 and 3.1.8.

The deflection of longitudinal section for a particular load level is shown in Fig. 3.1.9.

3.2 Prestressed Concrete Folded Plates:

The load-deflection curves for different load levels for different points on the folded plate were shown in Figs. 3.2.1 to 3.2.8. From Fig. 3.2.1. the load deflection relation of mid point on the ridge C_M for different folded plates can be seen. Here also it is seen that the deflection of folded plates with ties is less than that without ties. The deflection after cracking is lesser for specimen with increasing number of ties as is expected. But in the elastic range it is seen

that the deflection of folded plate with three ties is slightly more than that of folded plate with two ties and folded plate with one tie. But this deflection is less than that of folded plate without any tie. The effect of prestressing can be seen from the load deflection curves. The sudden change in the load-deflection curves shows the effect of cracking. The load deflection curves for points A (or E) and B(or D) can be seen from Figs. 3.2.2 and 3.2.3 respectively. Fig. 3.2.2 shows the local effect of tie. As the tie for FP1 is at mid span the deflection of point A (or E) for this specimen is less than that of other specimens. Here also the deflection of folded plates with ties is less than that without ties. In the inelastic range the deflection of point A_M (or E_M) of FP3 is less than that of FP2. But in the elastic range it is seen that this deflection of FP3 is more than FP2 or FP1 as is observed in the case of C_M . From fig. 3.2.3 it can be seen that in the elastic range the deflection of FP2 is less than FP1 of the point B_M (or D_M). But the deflection of FP3 is more than that of these two specimens as in the other cases. But the deflection of all the folded plates with ties is less than that without any tie. In the inelastic range the deflection of FP3 is much less than FP2 and that of FP2 is less than FP1. The load-deflection curves for

the point on the ridge at one third span C_L (or C_R) can be seen from figure 3.2.4. It is seen that the deflection of all the folded plates with ties is less than that of folded plate without any tie. In the elastic range the deflection of point C_L (or C_R) of FP2 is less than that of FP1. But the deflection of FP3 is more than that of FP1 or FP2. But in the inelastic range the deflection of FP3 is much less than that of FP2 and that of FP2 is less than FP1.

The load-deflection curves for the point A (or E) at one third span is shown in figure 3.2.5. Here also the deflections of folded plates with ties are less than that of folded plate without ties. The deflection for FP2 is less than that of FP1 but they follow very closely. Again in the elastic range the deflection of folded plate FP3 is slightly more than that of FP2 or FP1. But in the inelastic range this is more. The load-deflection curves for the point B (or D) at one third span is shown in fig. 3.2.6. Here also the deflection of folded plates with ties is less than that without ties. Here also the deflection curves for FP2 and FP1 closely follow each other. In the elastic range the deflection of FP3 is more than that of FP2 or FP1. But in the inelastic range this is less.

The load-deflection v along the plate curves for mid span and quarter span are shown in figs. 3.27 and 3.2.8.

The deflection curves of longitudinal section for one particular load level is shown in Fig. 3.2.9.

The folded plate FP4 was tested with one, two and three ties respectively upto a load of 1820 kg. The load-deflection behaviour in the elastic range is shown in Fig. 3.2.10 to 3.2.13. The behaviour here also is similar to that of FP1, FP2 and FP3.

3.3 Effect of Prestressing the Ties:

The tie bars of the reinforced specimen FP2 were prestressed to study the effect of prestressing. The tie bar force versus the deflection for the points A or E and B or D at quarter and mid-spans are shown in Figs. 3.3.1 and 3.3.2. The tie bar force versus the vertical deflection of C at quarter and mid spans is shown in Fig. 3.3.3. It is seen that the deflection at mid span is more than that at the quarter span in all the cases.

The tie bar force versus the deflection of points A or E and B or D at quarter span and mid span is shown in Figs. 3.3.4 to 3.3.7. It is observed that in the initial stages the deflection is more for the specimen with more number of ties. This is because prestressing of more number of ties will have more effect on deflections. At a higher prestressing force the deflection of FP2 is more. This is because of the horizontal crack parallel to

the ridge formed in this speciman during prestressing the ties. The sudden change in the deflection curve of FP2 indicates the formation of this horizontal crack.

It was observed in all the cases that the prestressing force in the tie bars was slightly reduced with the application of load and with the increase of load. This is because the two edges of the folded plate moved in due to bending.

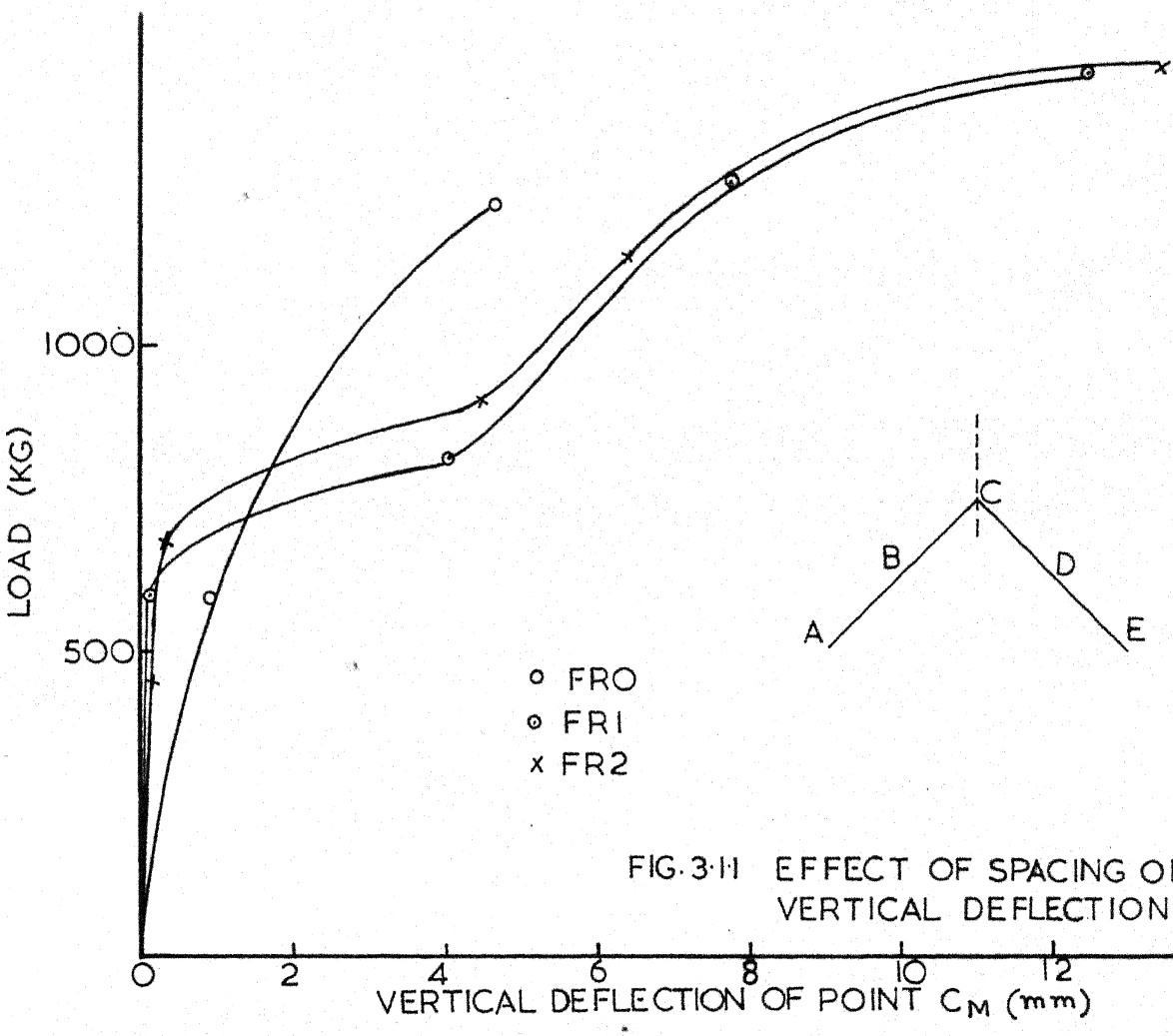


FIG. 3·1·1 EFFECT OF SPACING OF TIES ON VERTICAL DEFLECTION OF POINT C_M

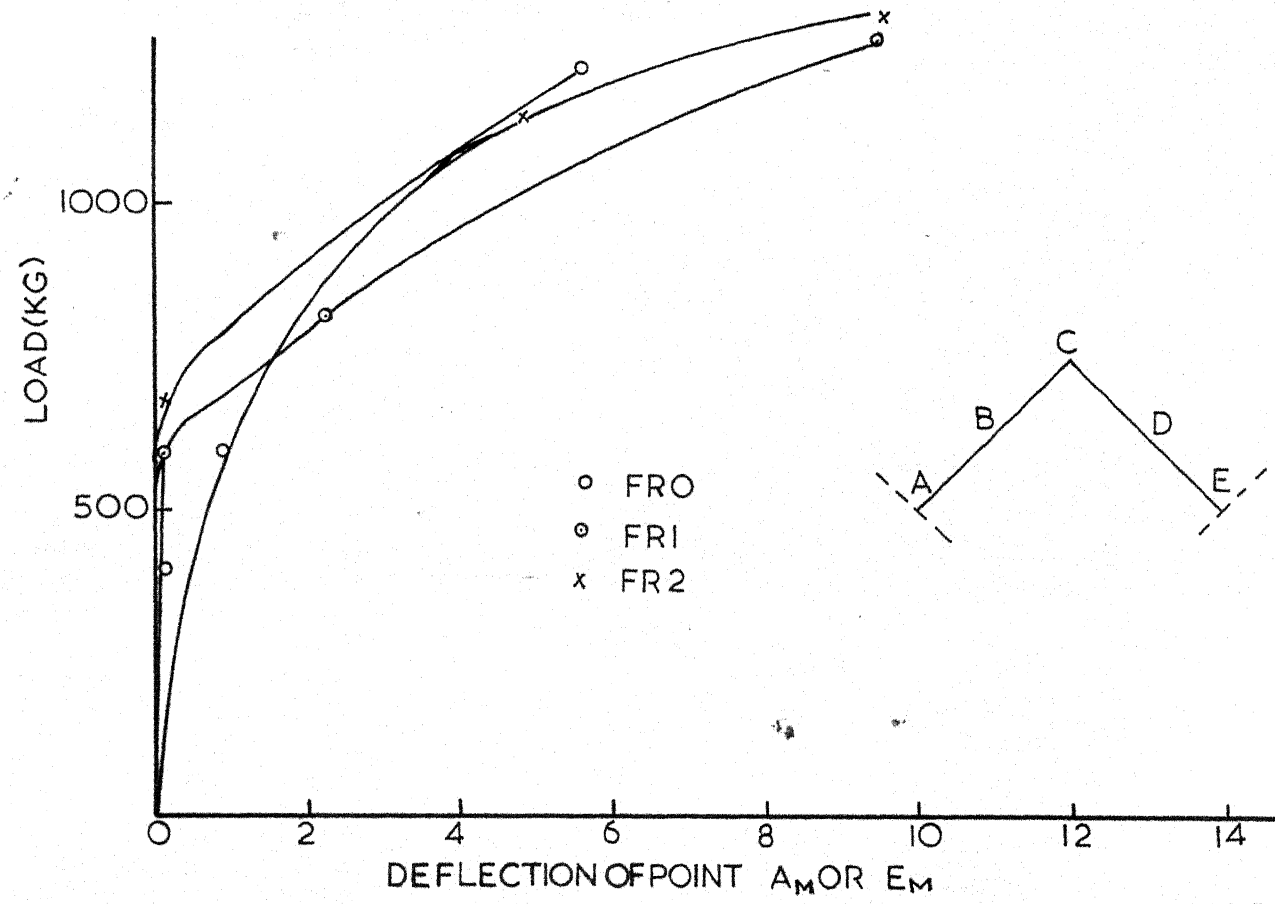


FIG. 3·1·2 EFFECT OF SPACING OF TIES ON DEFLECTION PERPENDICULAR TO PLATE OF POINT A_M OR E_M

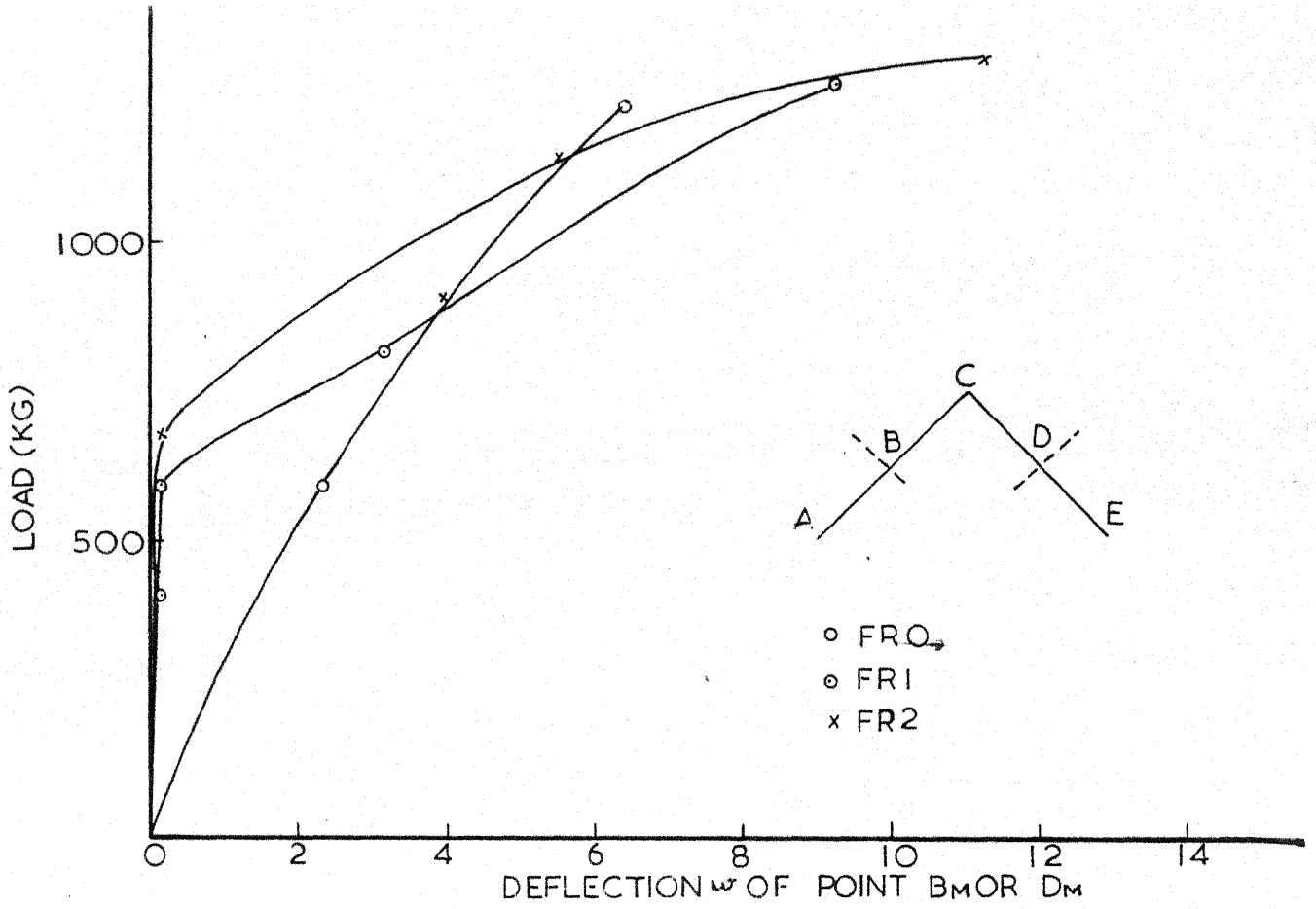


FIG.3·1·3 EFFECT OF SPACING OF TIES ON DEFLECTION PERPENDICULAR TO PLATE OF POINT B_M OR D_M

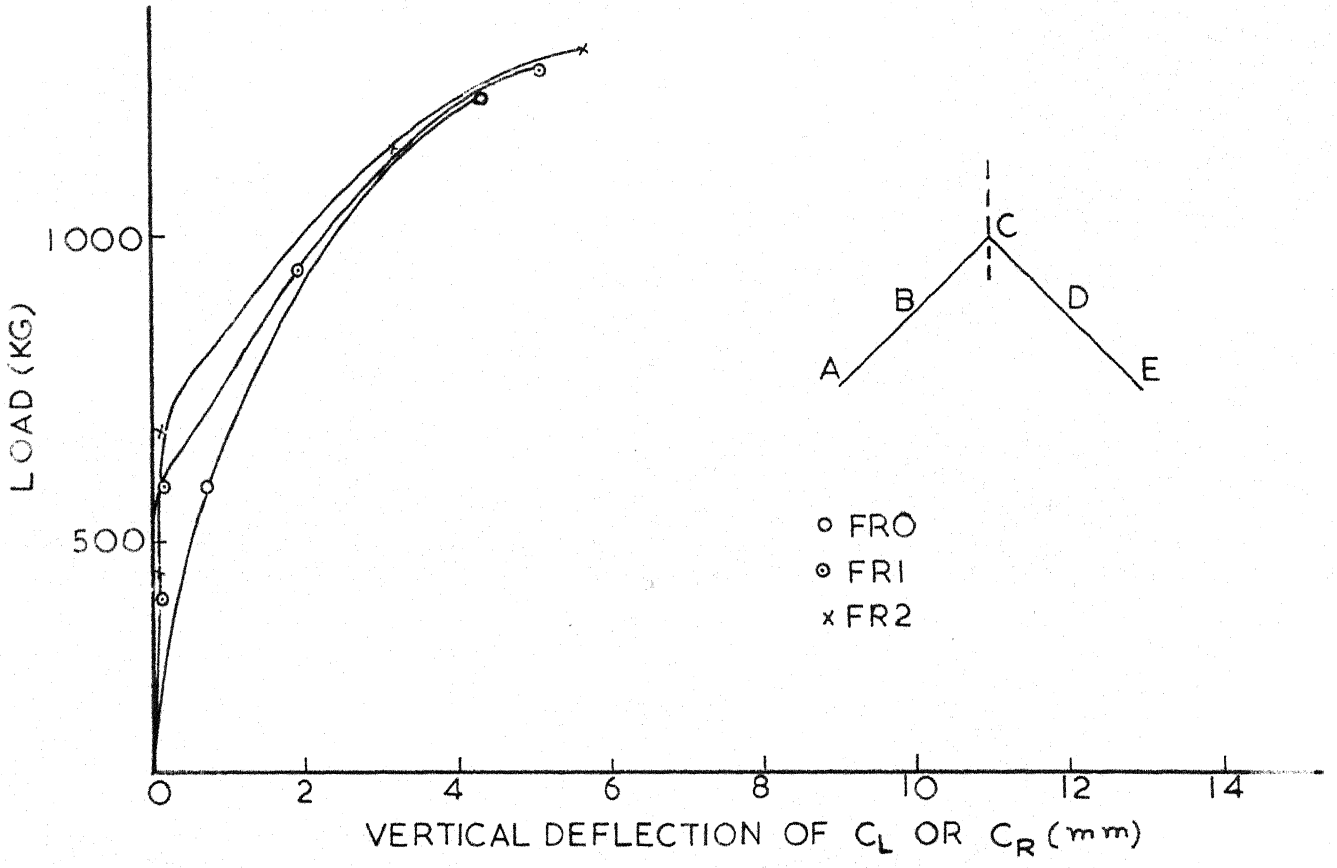


FIG 3·1·4 EFFECT OF SPACING OF TIES ON VERTICAL DEFLECTION OF POINT C_L OR C_R

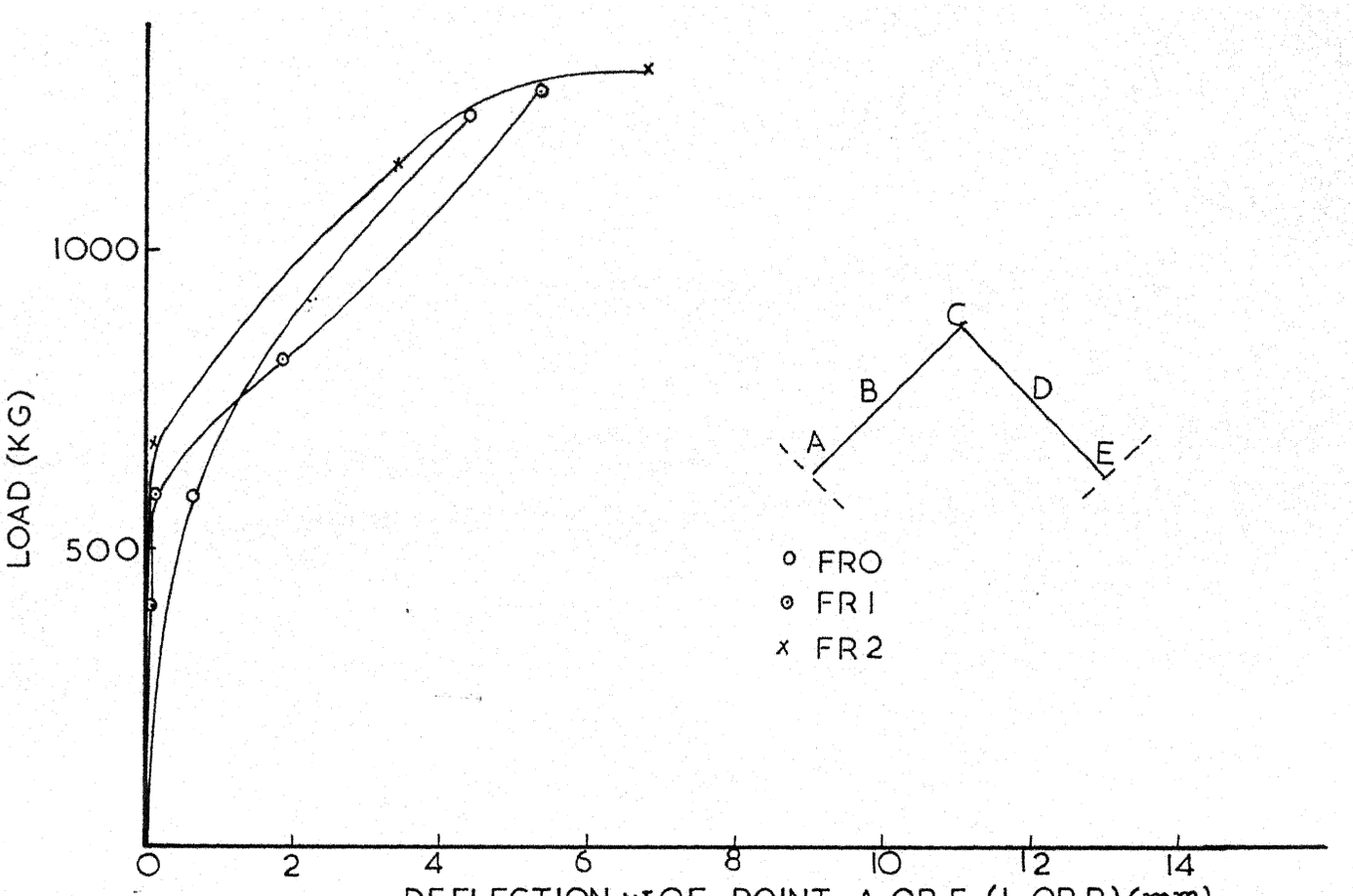


FIG.3·1·5 EFFECT OF SPACING OF TIES ON DEFLECTION PERPENDICULAR TO PLATE OF POINT A OR E (L OR R)

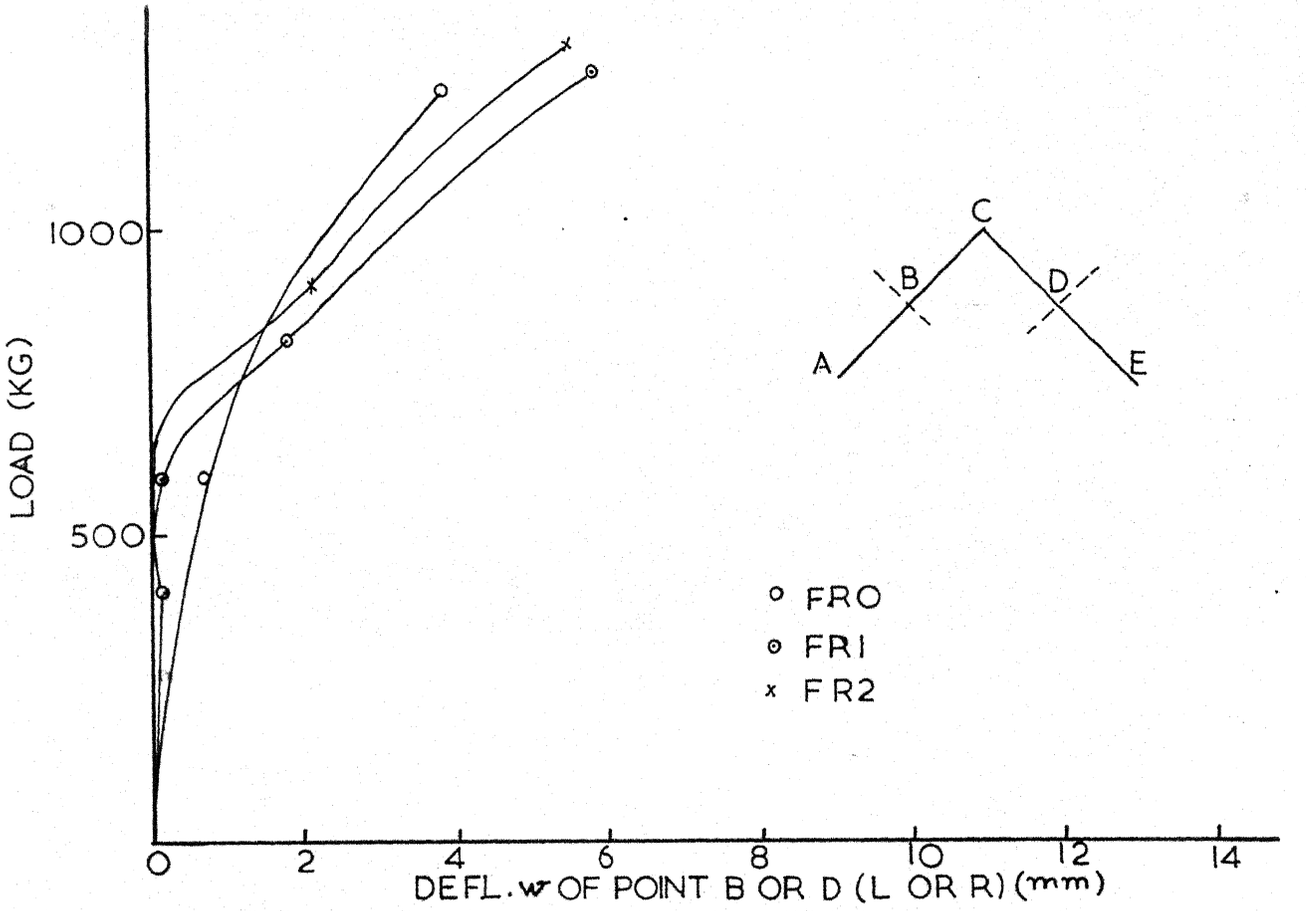


FIG.3·1·6 EFFECT OF SPACING OF TIES ON DEFLECTION PERPENDICULAR TO PLATE OF POINT B OR D (L OR R)

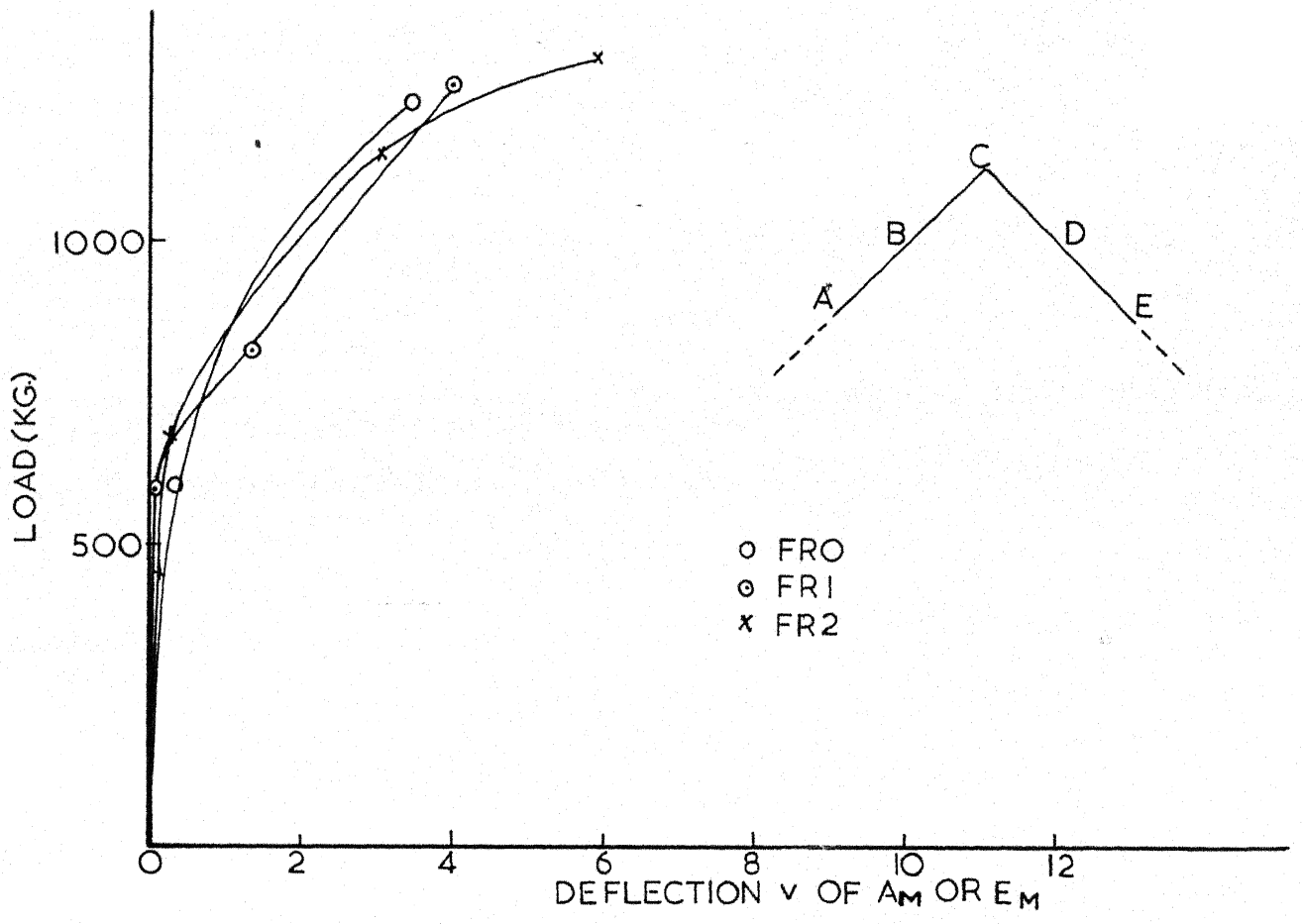


FIG. 3·1·7 EFFECT OF SPACING OF TIES ON DEFLECTION ALONG PLATE OF POINT A_M OR E_M

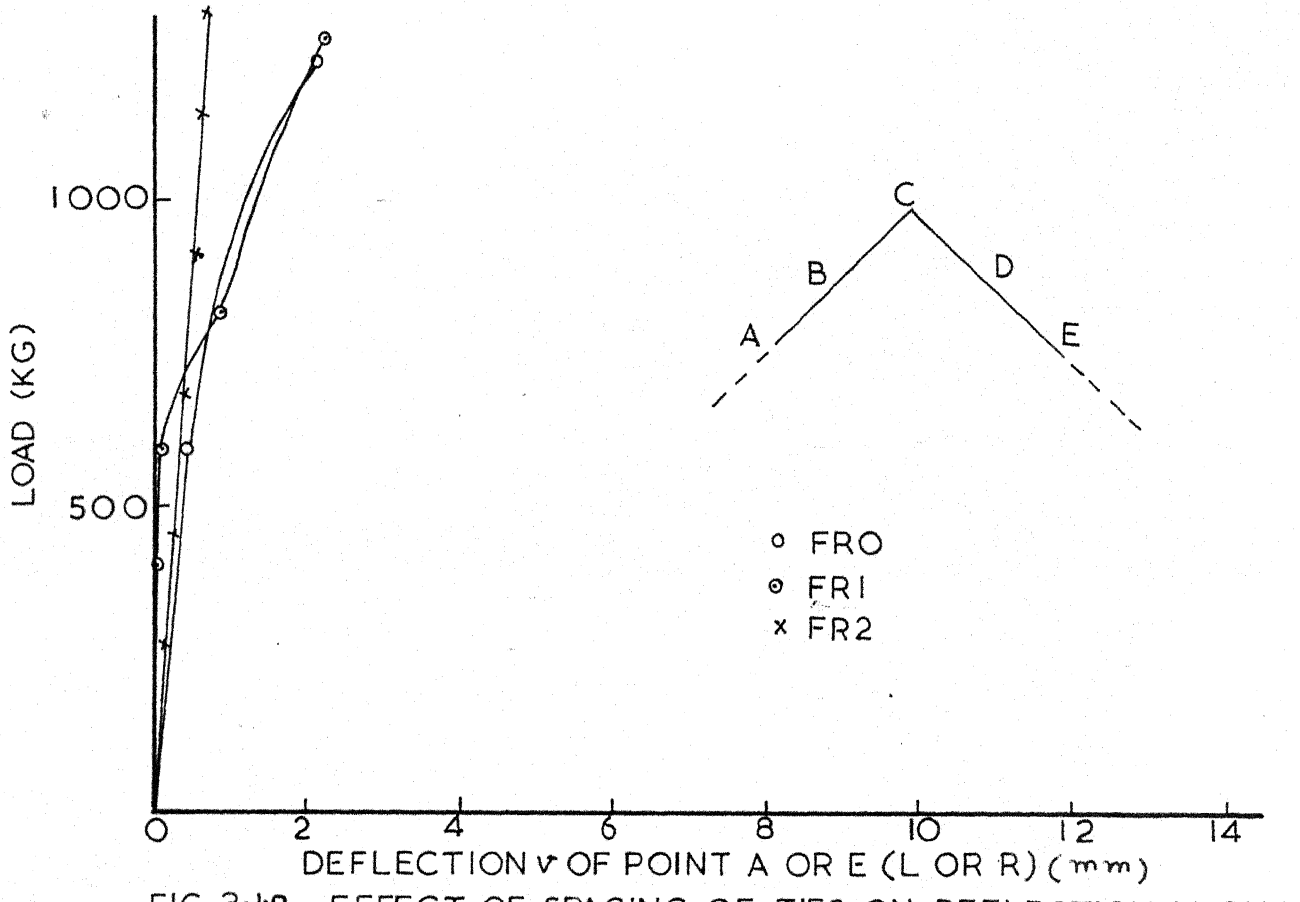


FIG. 3·1·8 EFFECT OF SPACING OF TIES ON DEFLECTION ALONG PLATE OF POINT A OR E (L OR R)

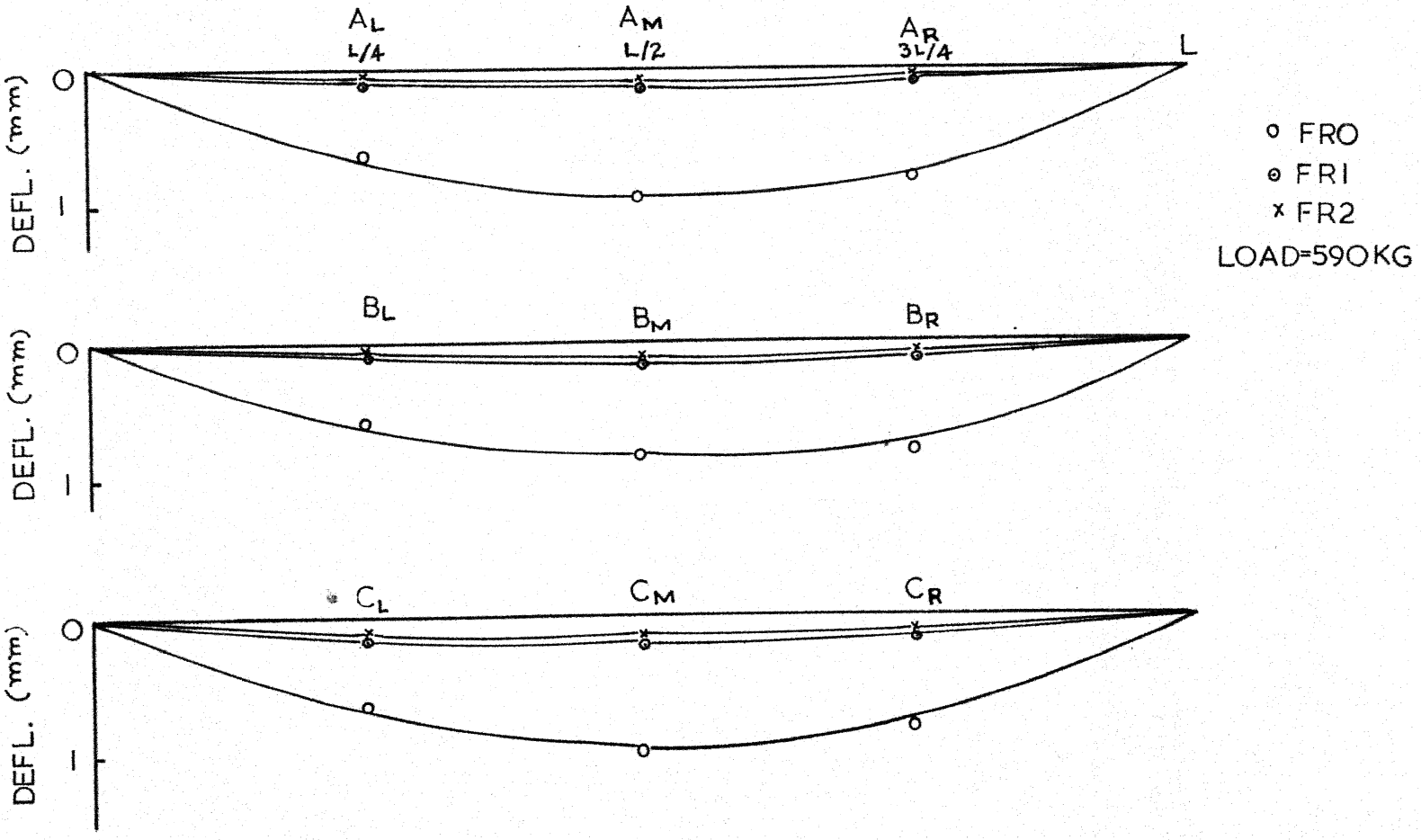


FIG. 3·1·9 DEFLECTION OF LONGITUDINAL SECTIONS

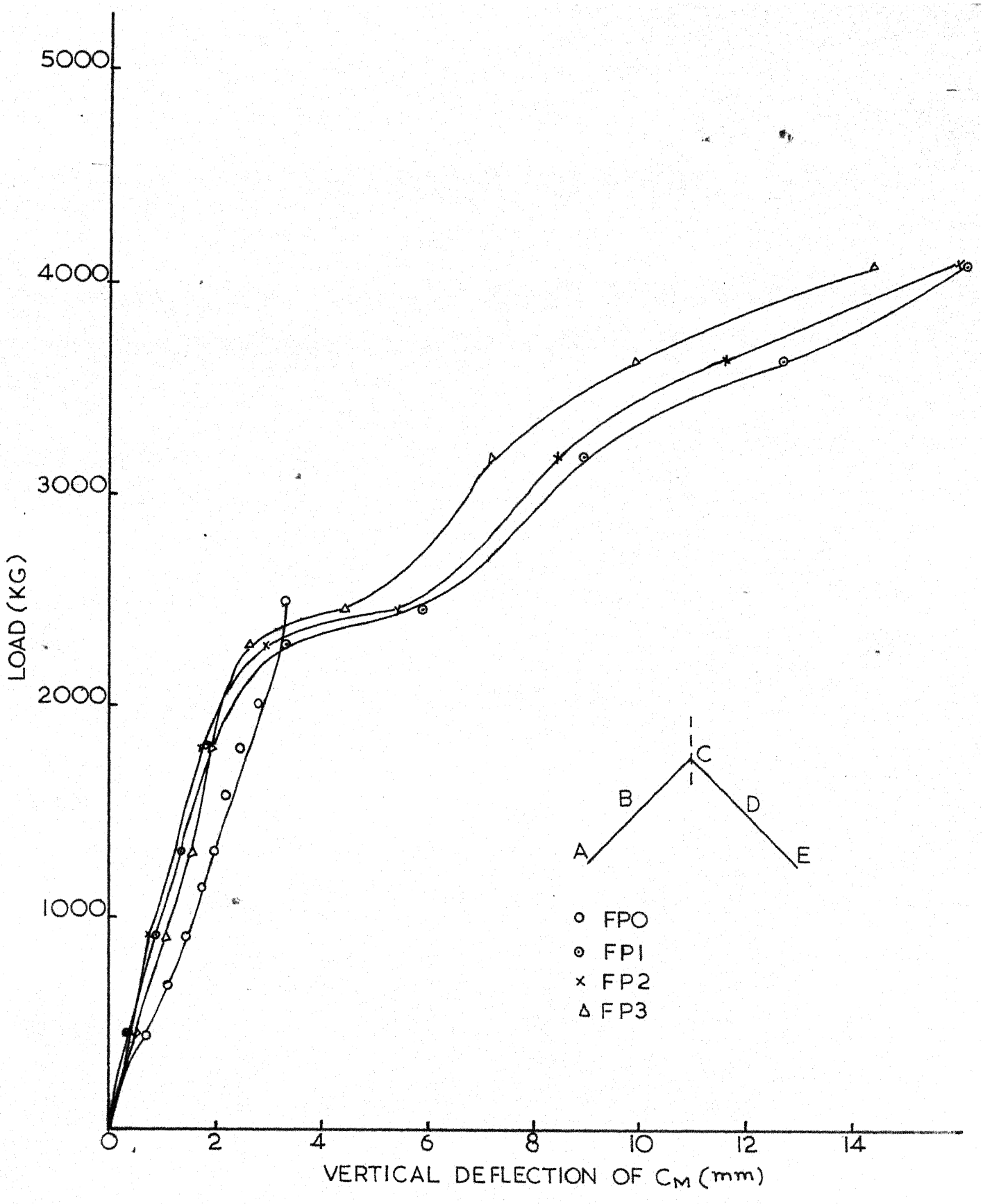


FIG. 3.2-1 EFFECT OF TIE SPACING ON VERTICAL DEFLECTION OF POINT C_M

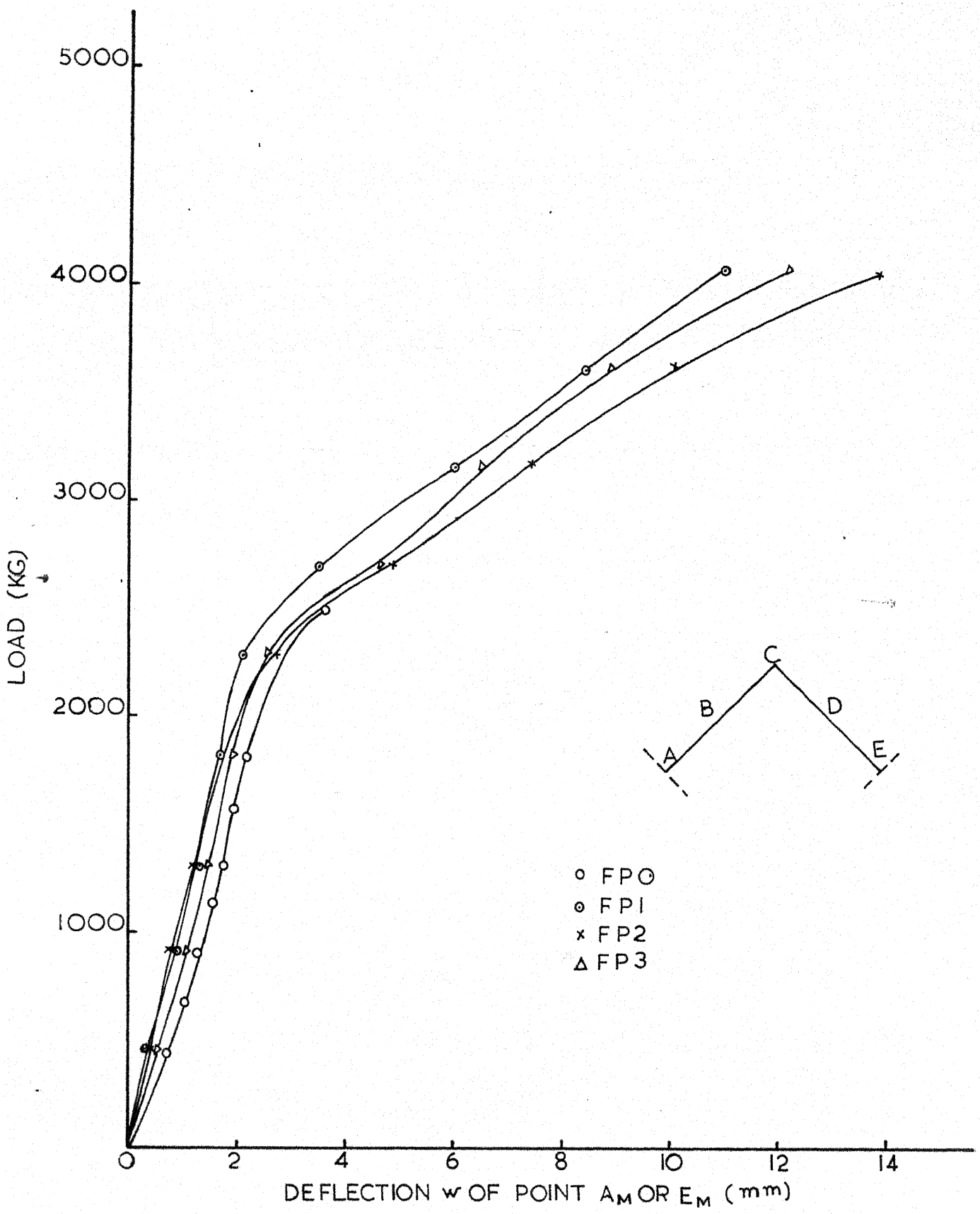


FIG.3·2·2 EFFECT OF TIE SPACING ON DEFLECTION PERPENDICULAR TO PLATE OF POINT A_M OR E_M

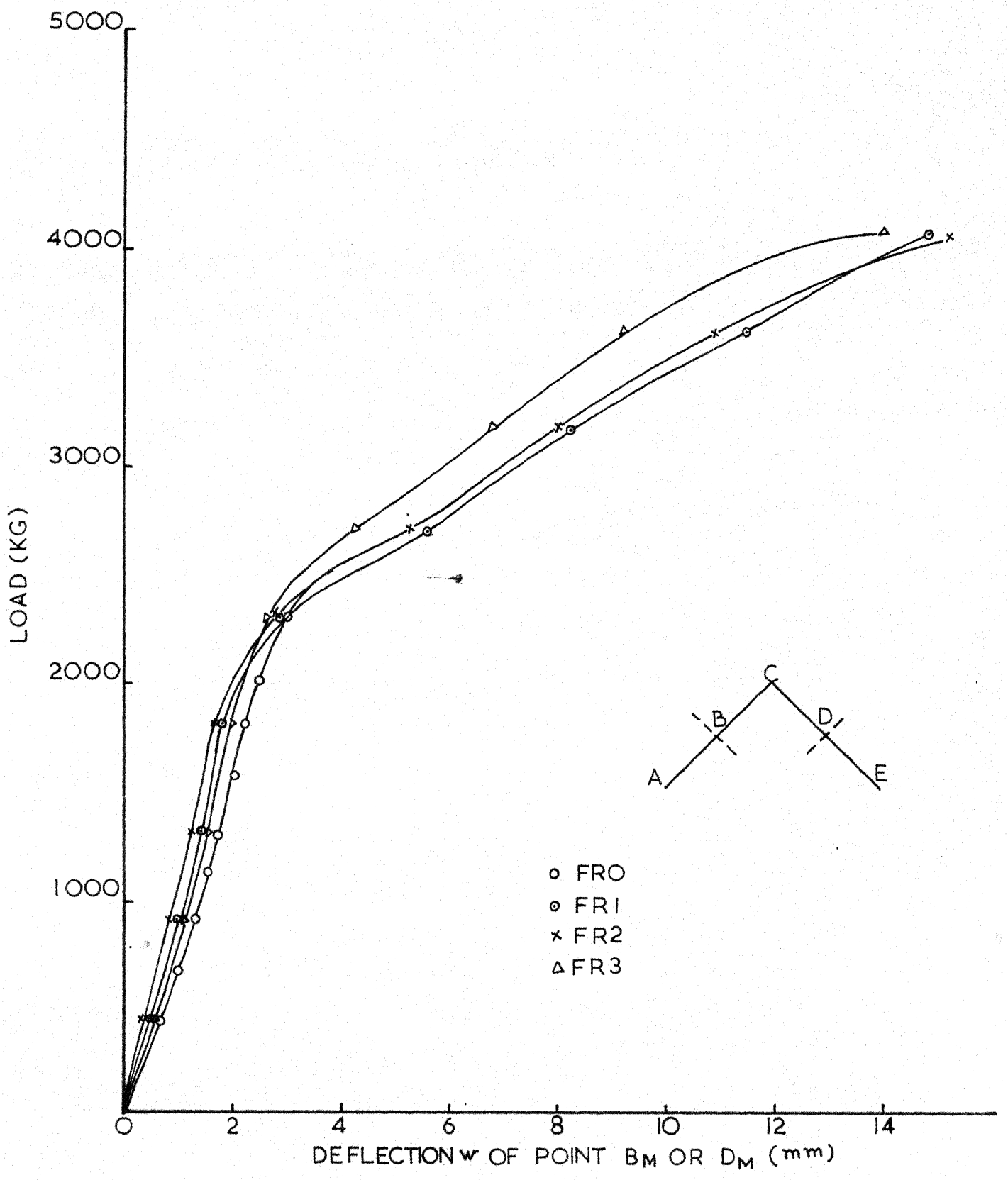


FIG.323 EFFECT OF TIE SPACING ON DEFLECTION PERPENDICULAR TO PLATE OF POINT B_M OR D_M

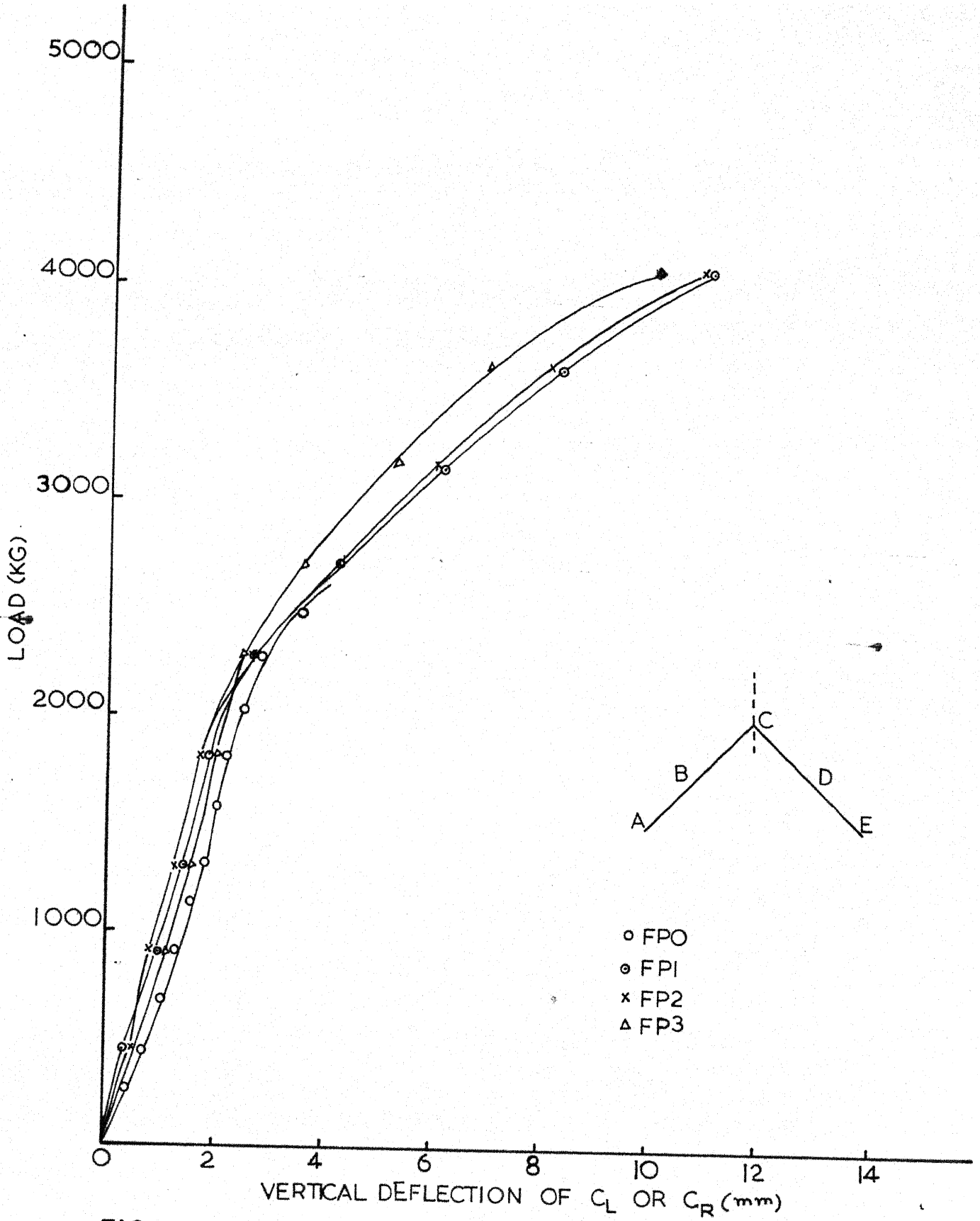


FIG.3·2·4 EFFECT OF TIE SPACING ON VERTICAL DEFLECTION OF POINT C_L OR C_R

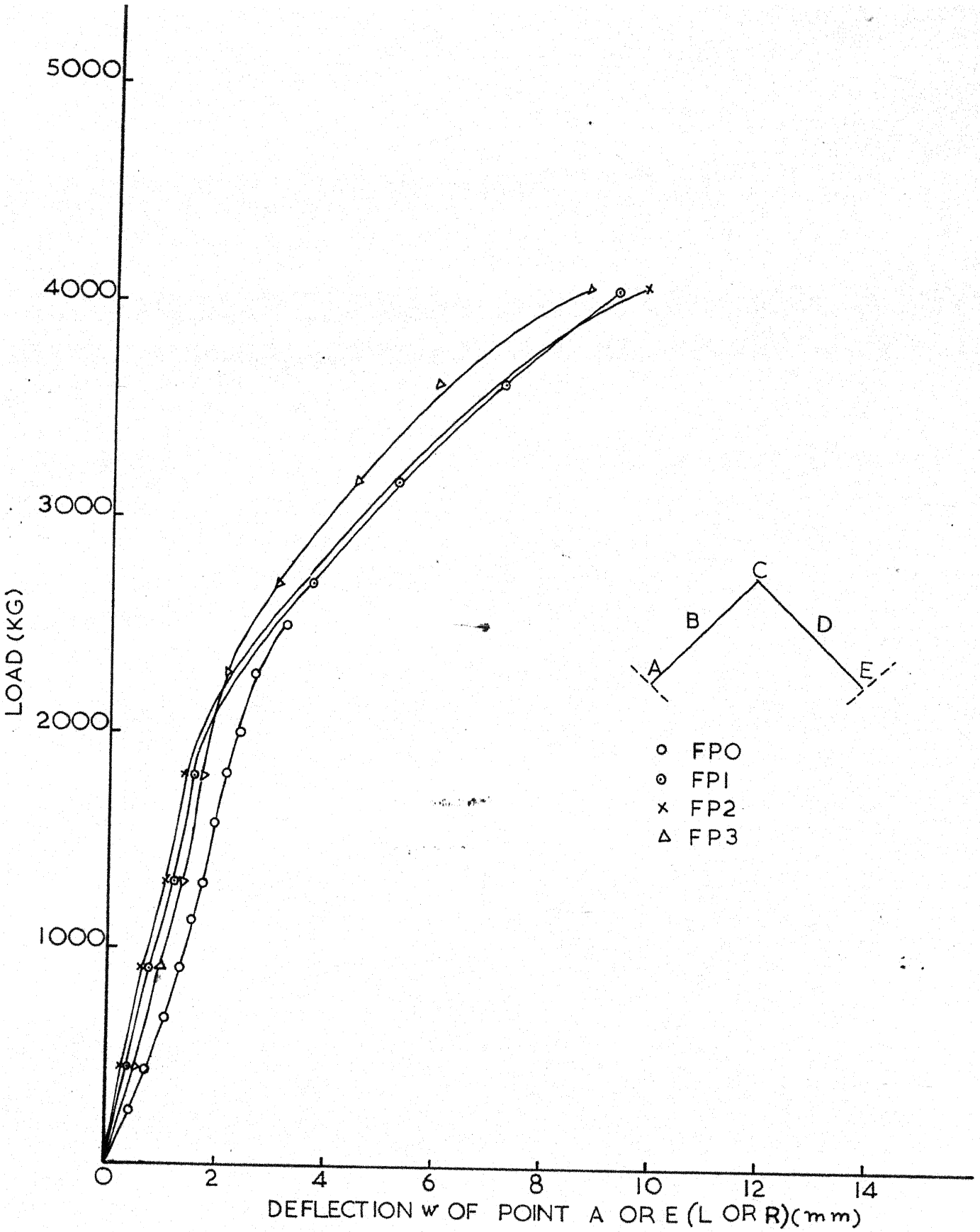


FIG.3.2.5 EFFECT OF TIE SPACING ON DEFLECTION PERPENDICULAR TO PLATE OF POINT A OR E (L OR R)

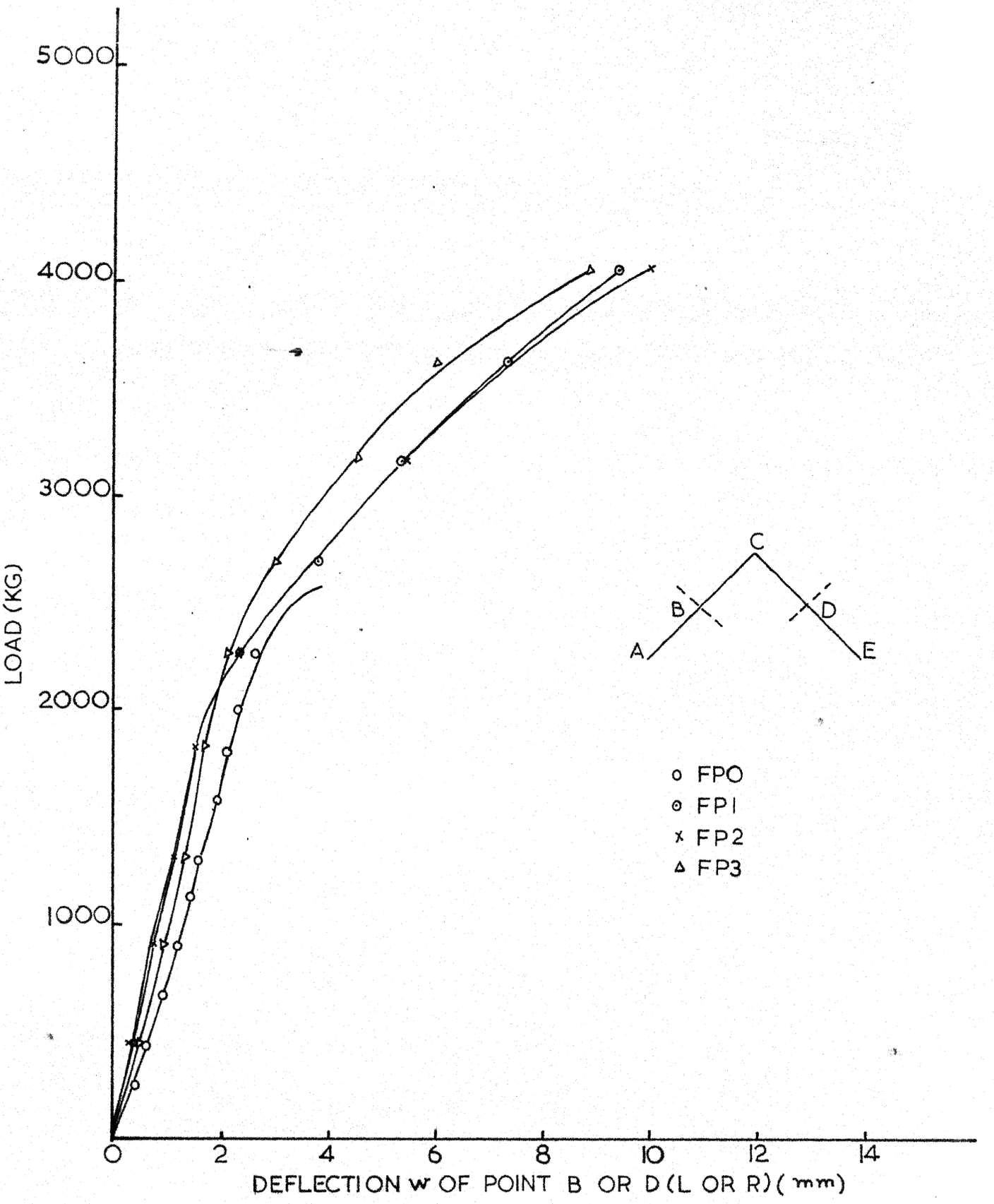


FIG.326 EFFECT OF TIE SPACING ON DEFLECTION PERPENDICULAR TO PLATE OF POINT B OR D (L OR R)

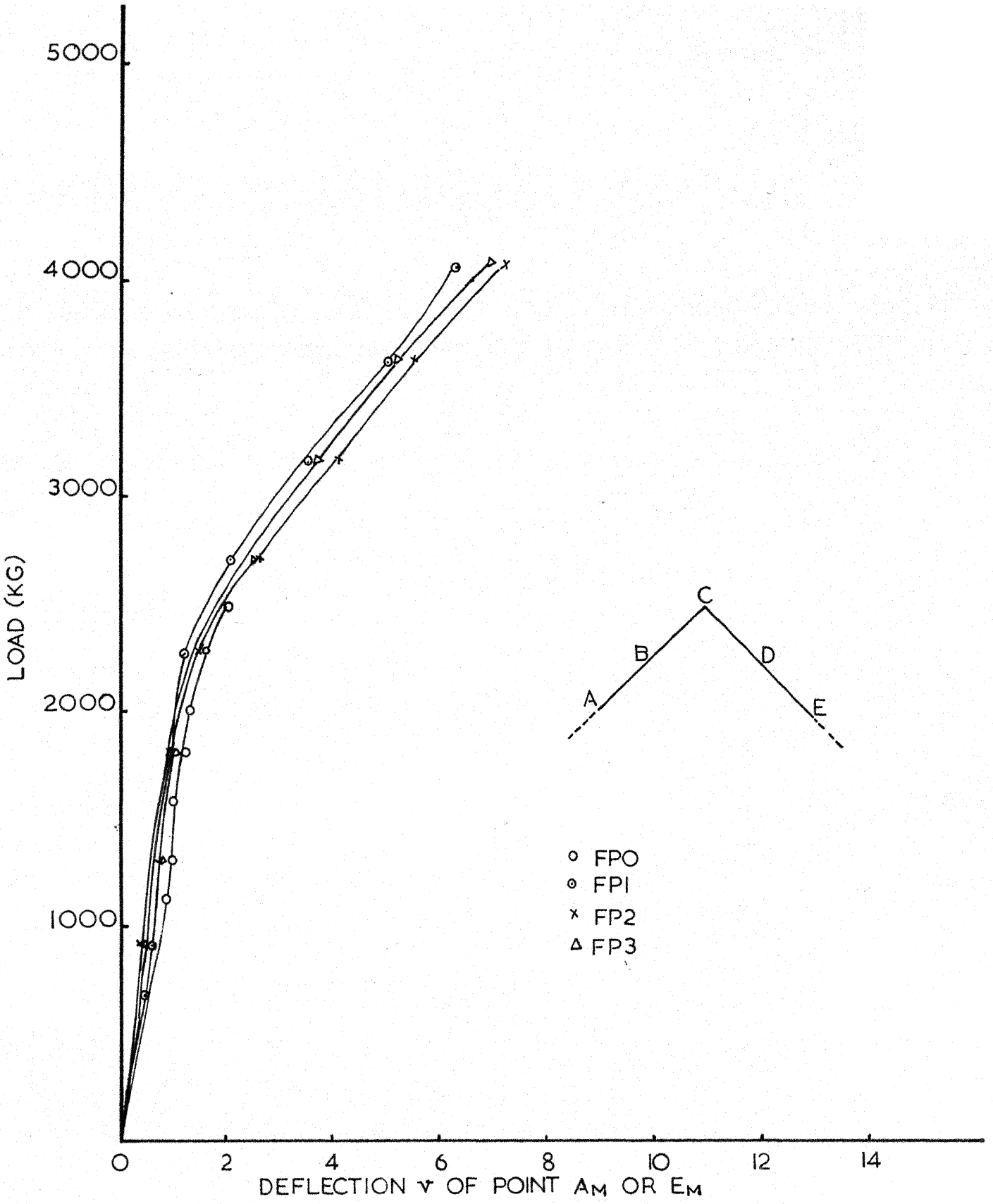


FIG. 3·2·7 EFFECT OF TIE SPACING ON DEFLECTION ALONG PLATE OF POINT AM OR EM

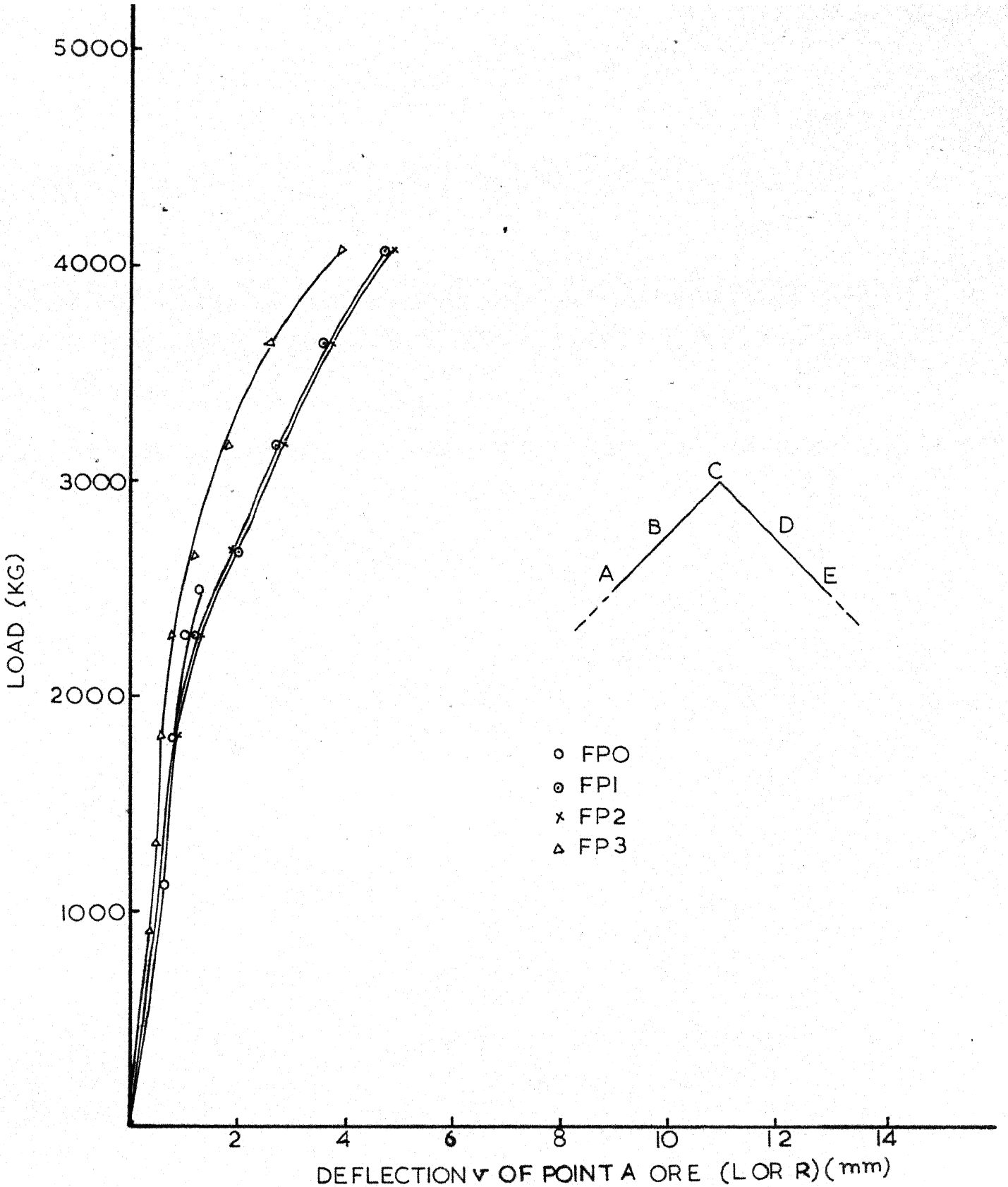


FIG. 3·2·8 EFFECT OF TIE SPACING ON DEFLECTION ALONG PLATE OF POINT A OR E (L OR R)

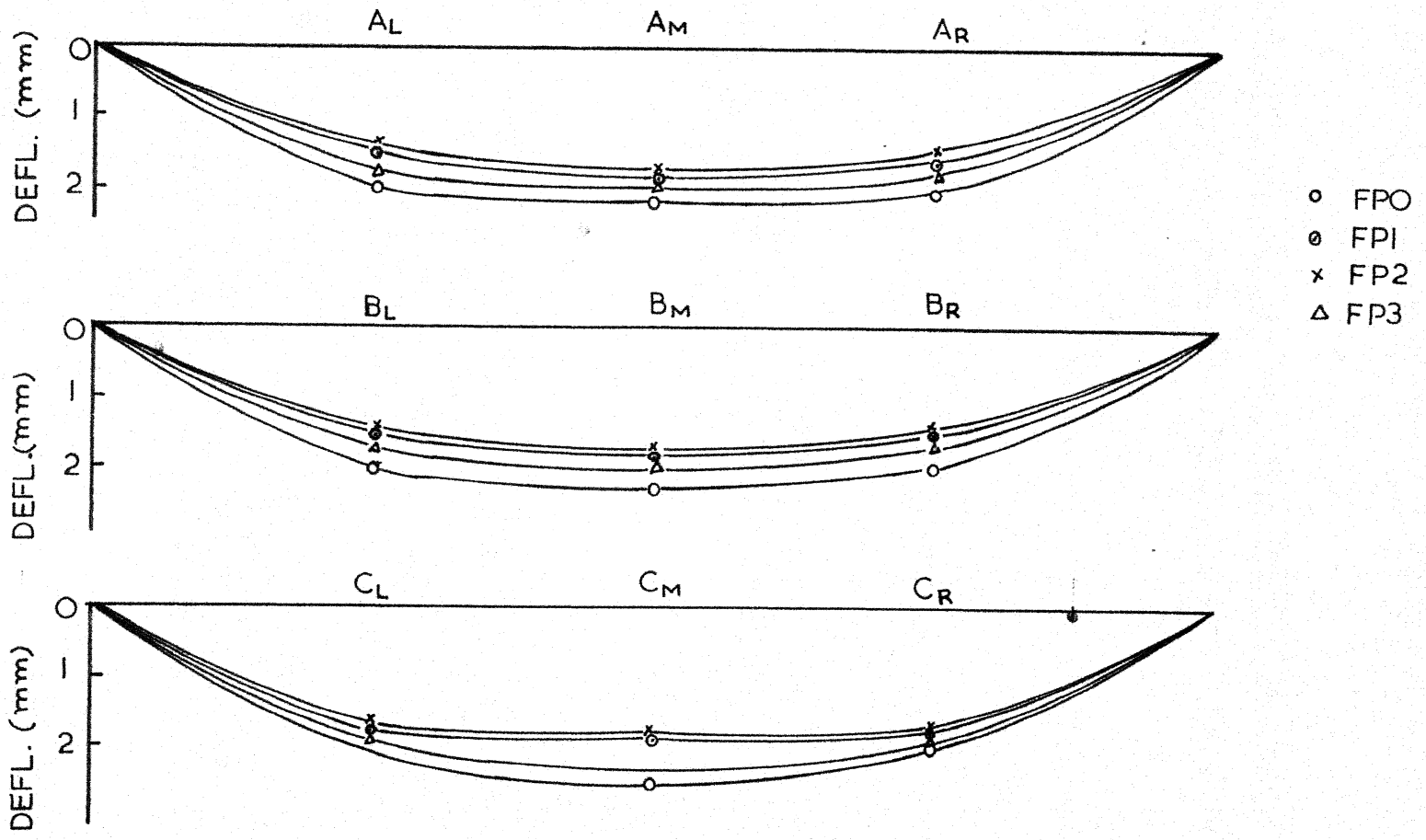
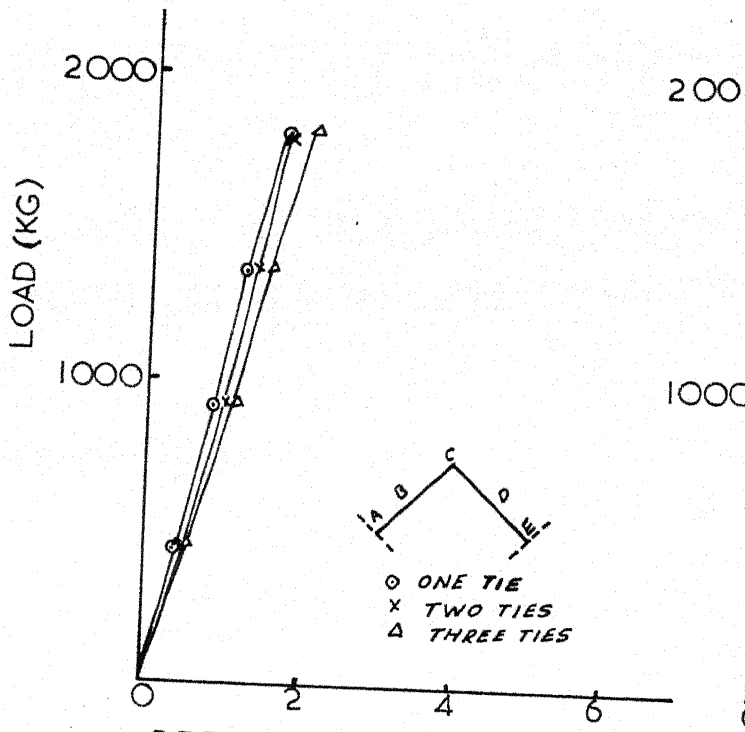
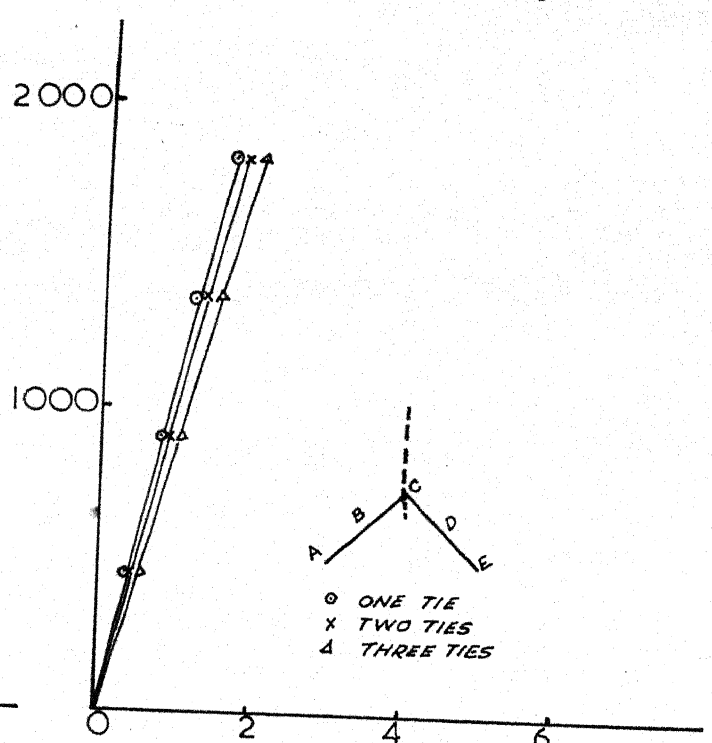


FIG. 3·2·9 DEFLECTION OF LONGITUDINAL SECTIONS FOR A LOAD OF 1820 KG



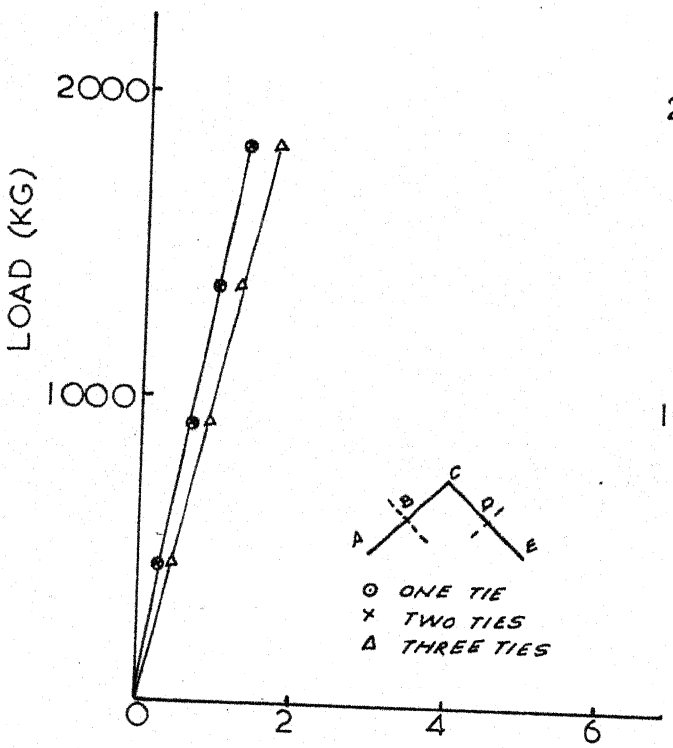
DEFL. W OF POINT A_M OR E_M (mm)

FIG. 3.2.10



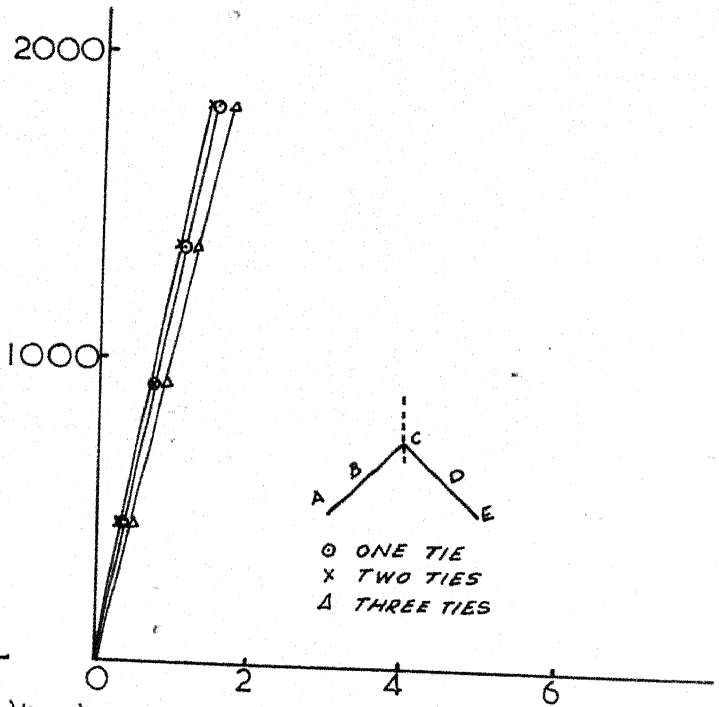
VERTICAL DEFL. OF C_M (mm)

FIG. 3.2.11



DEFL W OF POINT B OR D (L OR R) (mm)

FIG. 3.2.12



VERTICAL DEFL. OF POINT C (L OR R) (mm)

FIG. 3.2.13

LOAD - DEFLECTION CURVE OF FP4

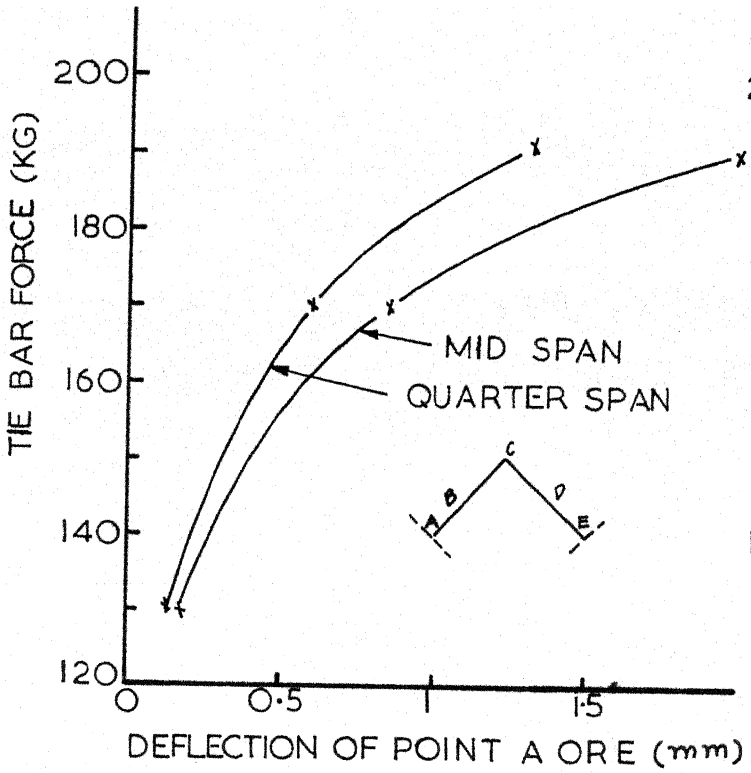


FIG. 3·3·1

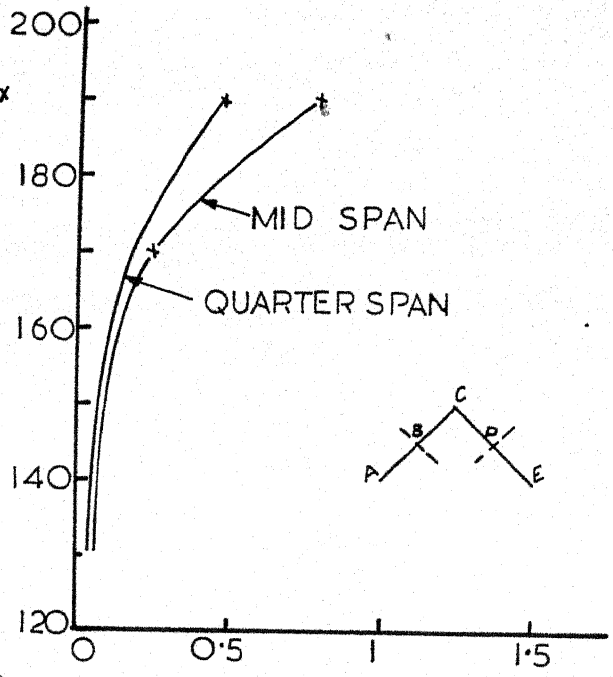


FIG 3·3·2

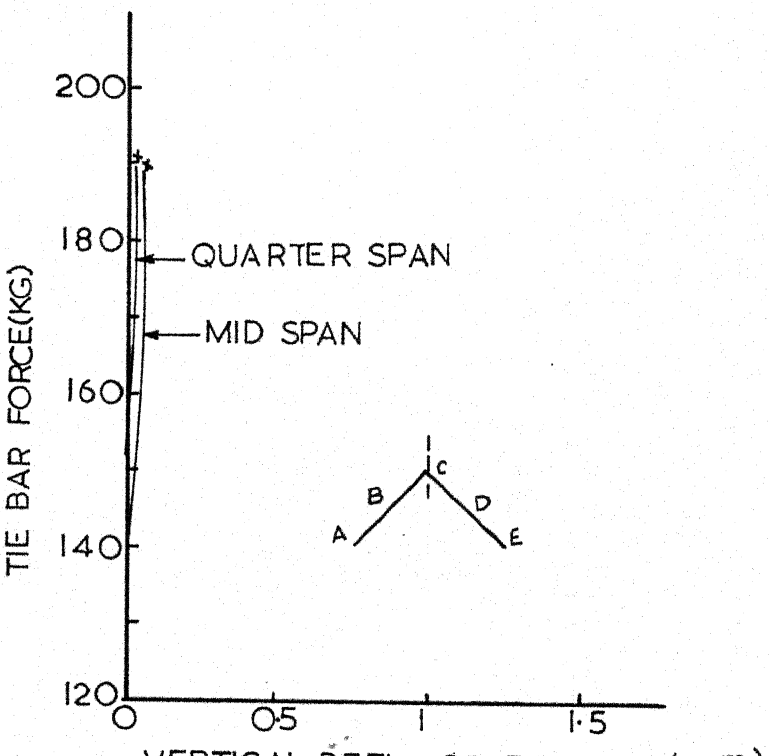
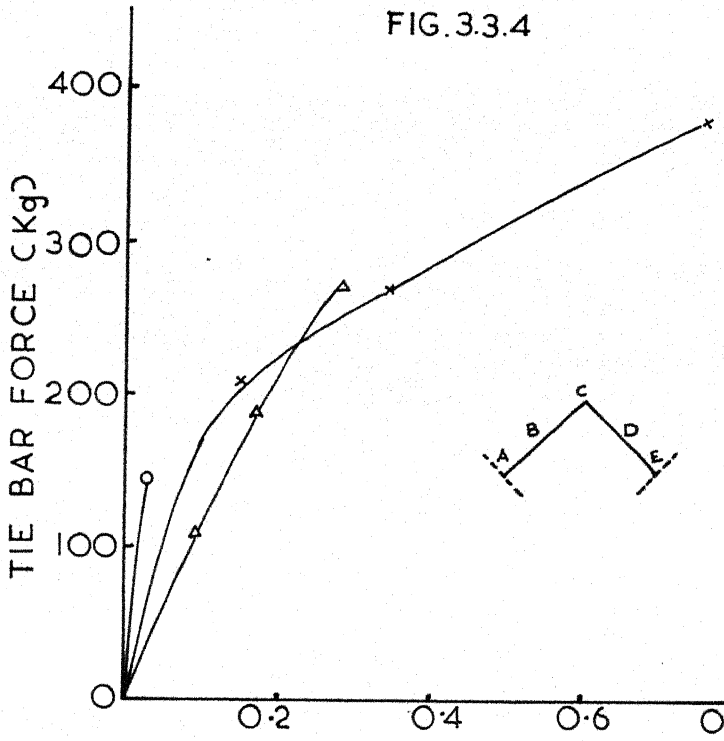


FIG. 3·3·3

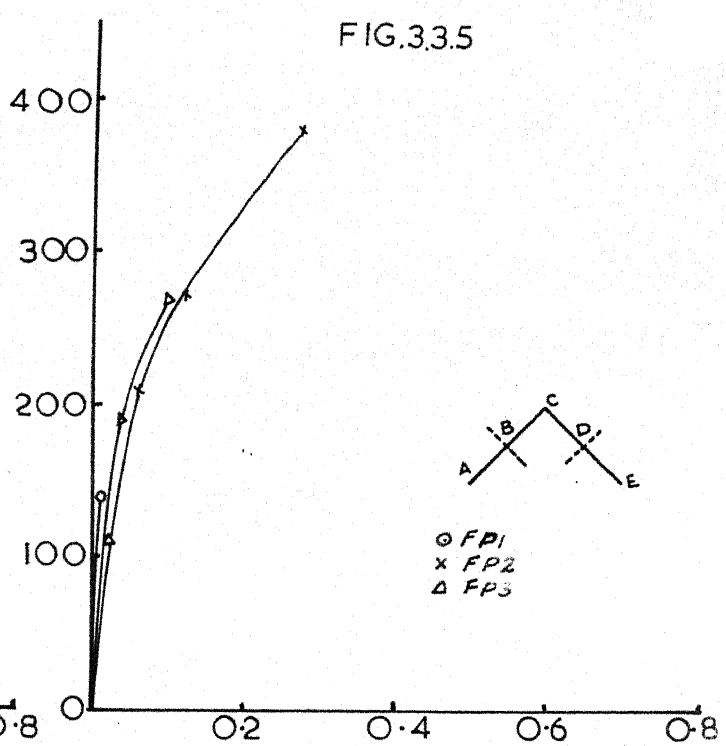
TIE BAR FORCE V_S DEFLECTION

FIG. 3.3.4



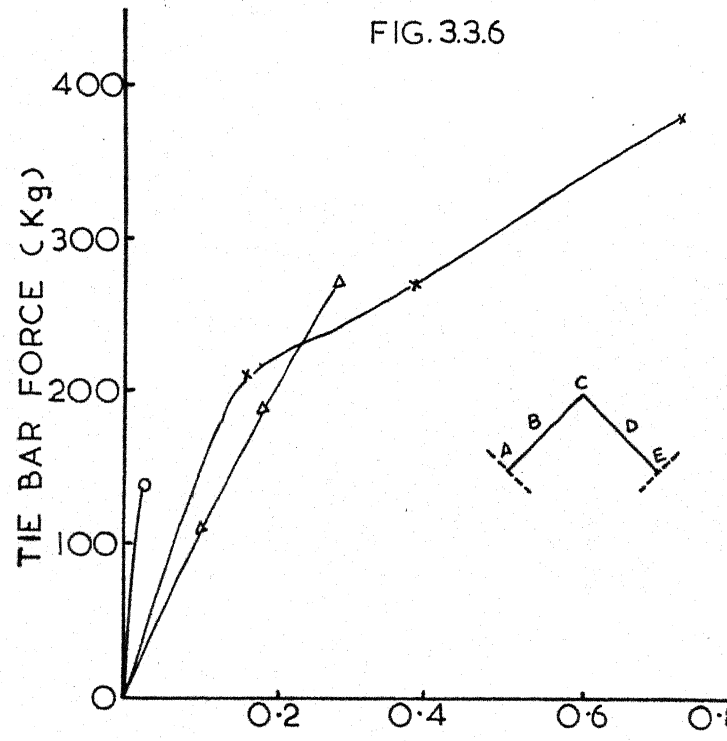
DEFL. OF POINT A OR E AT QUARTER SPAN (mm)

FIG. 3.3.5



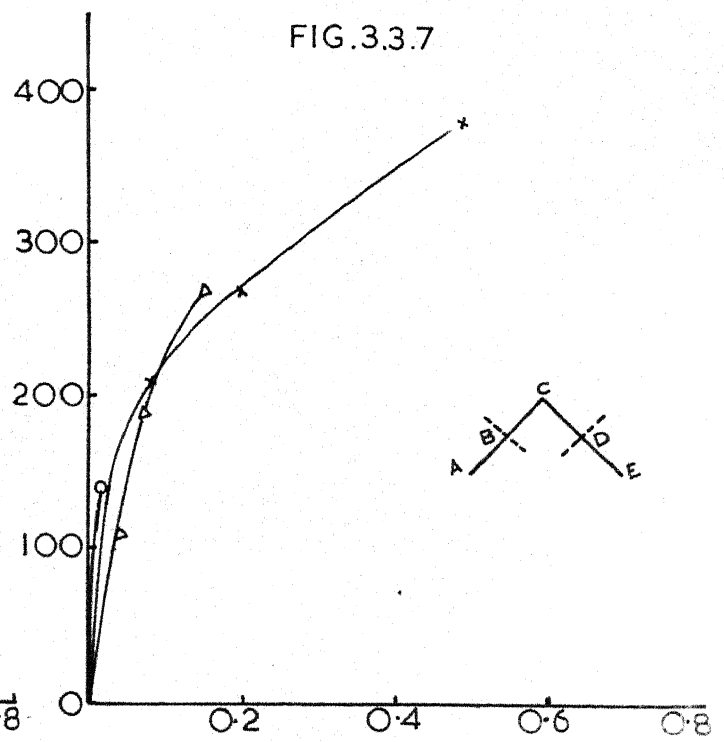
DEFL. OF POINT B OR D AT QUARTER SPAN (mm)

FIG. 3.3.6



DEFL. OF POINT A ORE AT MID-SPAN (mm)

FIG. 3.3.7



DEFL. OF POINT B OR D AT MID SPAN (mm)

TIE-BAR FORCE Vs DEFLECTION

3.4 Ultimate Strengths:

3.4.1 R.C.C. Folded Plates: The ultimate strength of the folded plates was calculated using the equivalent rectangular theory. The equivalent area and the equivalent rectangle width were calculated. Percentage of tension reinforcement and compression reinforcement in the case of reinforced specimens were calculated. Compression reinforcement was neglected as $(p - p')$ was less than p_{max} in all the three cases. The ultimate moment for the reinforced specimens was calculated using the formula[7]

$$M_u = P_{su} \left(d - \frac{0.6\, P_{su}}{b\, f'_c} \right)$$

where

M_u = ultimate moment

P_{su} = ultimate load taken by steel

b = equivalent rectangle width

f'_c = 4" cube strength of concrete.

All the specimens were loaded to failure while testing and the ultimate load was recorded. Table 3.4.1 shows the theoritical and experimental values of ultimate load.

TABLE 3.4.1: Ultimate Strength of Reinforced Concrete Folded Plates:

Specimen	Longitudinal Reinforcement	f'_c in kg/cm^2	P_u (theoritical) in kg.	P_u (experimental) in kg.
FR0	9 bars of 1/8 in. dia.	470	1740	1965
FR1	7 bars of 1/8 in. dia.	504	1060	1545
FR2	7 bars of 1/8 in. dia.	525	1060	1453

The theoritical values of ultimate load are less than the experimental values. This is most probably due to the ommission of the contribution of longitudinal reinforcement provided at the ends on either side, in the theoritical calculations.

The theoritical calculations are shown in Appendix.

3.4.2 Prestressed Concrete Folded Plates:

The ultimate strength of prestressed specimens also was calculated using the equivalent rectangle theory. The equivalent area and the equivalent rectangle width were calculated. The ratio $\frac{p\ f_{su}}{f'_c}$ was found to be less than 0.3 in all the cases and therefore the following formula was used.

$$M_u = A_s\ f_{su}\ \left(d - \frac{a}{2}\right)$$

where $$a = \frac{A_s\ f_{su}}{0.85\ f'_c\ b}$$

All the specimens were loaded upto failure and the ultimate load was noted in each case. Table 3.4.2 shows the theoritical and experimental values.

TABLE 3.4.2: ULTIMATE STRENGTH OF PRESTRESSED CONCRETE FOLDED PLATES

Specimen	Reinforcement	f'_c	P_u (Theoritical)	P_u (Exp.)
		kg./cm^2	Kg.	Kg.
FP0	4 nos. 5 mm. dia. high tensile steel wires.	470	4460	4500
FP1	" " "	500	4500	4270
FP2	" " "	530	4560	4270
FP3	" " "	495	4496	4770
FP4	" " "	444	4380	4900

The theoritical calculations of ultimate strength are shown in Appendix.

3.5 Secondary Stresses from Experimental Observation:

Transverse ties were provided at intermediate points of the span. Due to the provision of these transverse ties secondary bending moments both in the transverse and longitudinal directions are induced. The secondary stresses caused by these bending moments are calculated. Considering a strip of unit width in the transverse direction it has bent in the plane of the plate. The plane of the plate is different from the

centroidal plane of the folded plate. Plane of the plate is along the direction of the individual plates. Similarly considering a strip of unit width in the longitudinal direction this also has bent in the longitudinal direction along the plane of the individual plate.

3.5.1 Secondary Stresses of Reinforced Concrete Folded Plates:

Secondary transverse bending moments at points B and C of quarter span and mid span were calculated using the experimental values of deflections. A second degree polinomial was assumed for the deflection curve. The deflection at any point x is given by

$$y = C_1 x^2 + C_2 x + C_3$$

From the known values of y at A, B and C the constants C_1, C_2 and C_3 were evaluated. Then the secondary transverse bending moment is calculated using

$$M = EI \frac{d^2y}{dx^2}$$

The secondary transverse stress is given by $\sigma = \frac{MC}{I}$. This gives only an approximate idea of the secondary transverse stress.

The secondary transverse stresses of the reinforced concrete folded plates with ties for a load of 590 kg. are tabulated in table 3.5.1(a).

TABLE 3.5.1(a): SECONDARY TRANSVERSE STRESSES OF REINFORCED CONCRETE FOLDED PLATES WITH TIES FOR A LOAD OF 590 KG.

	Secondary transverse stress of quarter span at B or C kg./cm^2	Secondary transverse stress of Mid-span at B or C kg./cm^2
FR1	7.38	17.15
FR2	2.46	7.38

Secondary longitudinal moments were calculated at quarter span and mid-span using the finite difference approximation of the experimental deflection curves. The program was run of on IBM 7044 digital computer. Using the known ordinates of the deflection curves the required ordinates at the intermediate points were calculated by Newton's interpolation formula. The input data consisted of the known deflection ordinates at the supports, quarter spans and mid-span. Using the calculated intermediate ordinates the secondary longitudinal bending moments for unit width of the plate were computed using

$$M_i = \frac{EI}{h^2} (w_{i-1} - 2 w_i + w_{i+1})$$

where E = modulus of elasticity of concrete

I = moment of inertia of unit width of plate

h = finite difference sub-interval.

w_{i-1} = deflection at the point i-1

w_i = deflection at the point i

w_{i+1} = deflection at the point i + 1.

From the calculated secondary longitudinal moments the secondary longitudinal stresses at quarter span and mid-span were calculated using $\sigma = \frac{MC}{I}$. The secondary longitudinal stresses of reinforced concrete folded plates for a load of 820 kg. are tabulated in table 3.5.1(b).

TABLE 3.5.1(b): SECONDARY LONGITUDINAL STRESSES OF REINFORCED CONCRETE FOLDED PLATES FOR A LOAD OF 820 KG.

	Secondary longitudinal stress at quarter span (kg./cm^2)			Secondary longitudinal stress at mid-span (kg./cm^2)		
	Near A	At B	At C	Near A	At B	At C
FR1	2.850	1.330	3.30	1.255	1.52	4.74
FR2	0.841	0.374	0.14	1.590	3.37	7.70

3.5.2 Secondary stresses of prestressed concrete folded plates:

The secondary transverse stresses of prestressed concrete folded plates were calculated similar to that in the case of reinforced concrete folded plates. The secondary transverse stresses of prestressed concrete folded plates with ties for an applied load of 910 kg. are tabulated in table 3.5.2(a).

TABLE 3.5.2(a): Secondary transverse stresses of prestressed Concrete Folded Plates for a Load of 910 kg.

	Secondary transverse stress of quarter span at B or C $Kg./cm^2$	Secondary transverse stress of mid-span at B or C $Kg./cm^2$
FP1	9.85	48.0
FP2	7.40	23.1
FP3	3.69	29.6

The secondary longitudinal moments of the prestressed concrete folded plates also were computed similar to that of reinforced concrete folded plates. The secondary longitudinal stresses were calculated using $\sigma = \frac{P}{A} \pm \frac{MC}{I}$ where P is the axial prestressing force. The secondary longitudinal stresses of prestressed concrete folded plates with ties for a load of 1820 kg. are tabulated in table 3.5.2(b).

TABLE 3.5.2(b): Secondary longitudinal stresses of prestressed concrete folded plates for a load of 1820 kg.

	Secondary longitudinal stress at quarter span (kg./cm2)			Secondary longitudinal stress at mid-span (kg./cm2)		
	Near A	At B	At C	Near A	At B	At C
FP1	21.49	22.93	21.23	56.18	51.69	48.51
FP2	32.33	32.89	24.83	50.34	45.57	57.40
FP3	34.23	32.93	26.03	45.81	46.41	56.93

3.6 Cracking Pattern and Mode of Failure:

The cracking pattern at different load levels was observed and sketched. Fig. 3.6.1 shows the cracking pattern of reinforced concrete specimens and figure 3.6.2 shows that of prestressed concrete specimens. In the case of FR0 first cracks were observed at a load level of 1410 kg. cracks propagated with increasing load. The cracking pattern for this specimen at a load of 1410 kg. and at ultimate is shown in Fig. 3.6.1(a) and (b). In the case of FR1 a hairline crack started at a load of 730 kg. near the bar location. Another crack was observed at a load of 820 kg. These cracks propogated with increase of load and further cracks were developed near ultimate. The cracking pattern for this specimen is shown in Fig. 3.6.1(c), (d), (e) and (f). The cracking pattern

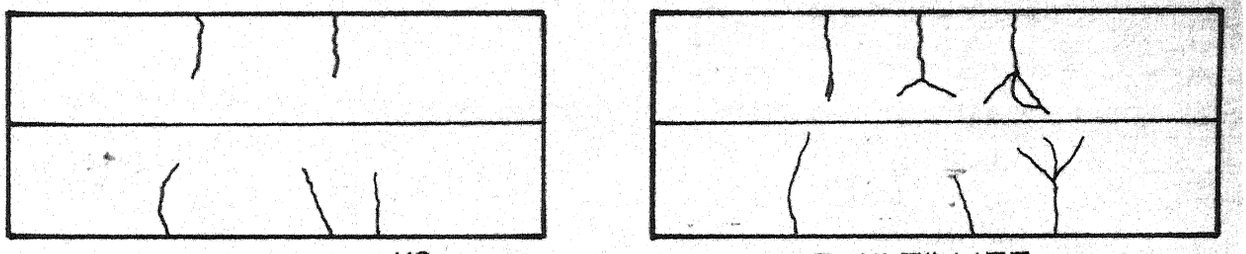
for FR2 is shown in fig. 3.6.1(g) and (h). A small hair-line crack was observed at a load of 910 kg. Further cracks developed near the tie bar locations with the increase of load. For the specimen FR0 the first crack was formed at a higher load because it has extra reinforcement. It is observed that the cracking of the specimens FR1 and FR2 is not as extensive as FR0.

The cracks are tension flexure cracks similar to any beam under flexure. The specimen also failed by tension failure as it is underreinforced. The behaviour of the specimens with or without ties could be treated as a beam under flexure.

The cracking pattern of prestressed specimens is shown in Fig. 3.6.2. The formation and propogation of cracks at different loads was carefully observed. The cracking pattern of PP0 is shown in figure 3.6.2(a) to (d). Small hair-line cracks were observed at a load of 2460 kg. The further development and propogation of cracks can be seen from the figures. For the specimen PP1 first cracking was observed at a load of 2280 kg. on one side of the folded plate as shown in Fig. 3.6.2(e). Cracks on the other side of the folded plate also were observed at a load of 2710 kg. Inclined cracks were observed at a load of 4090 kg. near one end. The cracking pattern for this specimen is shown in figure 3.6.2(f) to (h). For the specimen PP2 an inclined crack near one end was

observed at a load of 2280 kg. Additional cracks near the tie bar locations and at the mid-span were observed at a load of 2710 kg. Another inclined crack near the other end was observed at a load of 3180 kg. A vertical crack in the diaphragms also was observed. At ultimate a crack parallel to the ridge was observed. The cracking pattern for FF2 is shown in Fig. 3.6.2(j) to (m). In the case of FF3 a hair-line crack near the central tie bar was observed at a load of 2280 kg. Further cracks near the other tie bar locations was observed at a load of 2710 kg. All the cracks formed were near the tie bar locations only. The cracking pattern for this specimen are shown in Fig.3.6.2 (n) to(r).

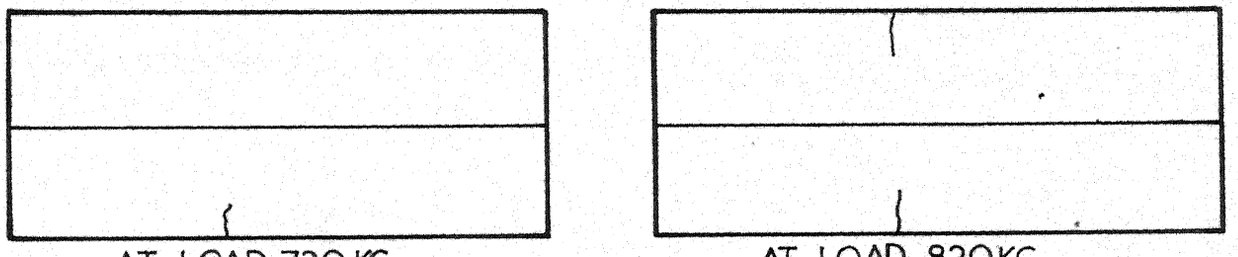
Eventhough the tension cracks are the first ones to appear during the test, all the specimens failed by crushing of the concrete at the ridge. The tension cracks only indicated an excessive deformation and did not lead directly to any failure as the specimens were over reinforced.



AT LOAD=1410 KG
(a)

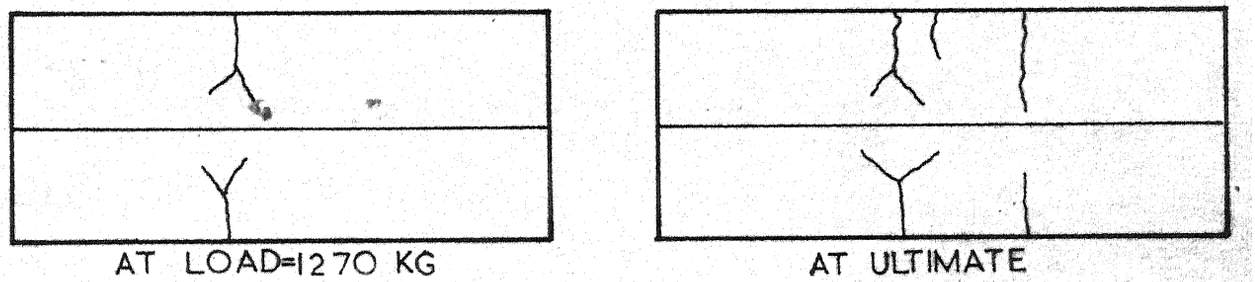
AT ULTIMATE
(b)

CRACKING PATTERN OF FRO



AT LOAD=730 KG
(c)

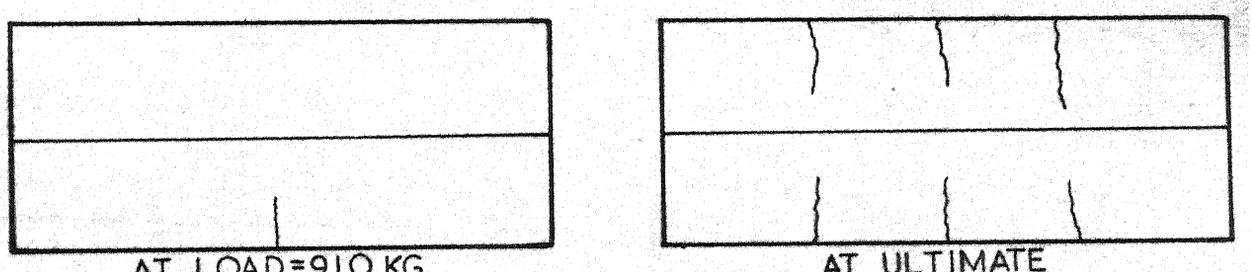
AT LOAD 820KG
(d)



AT LOAD=1270 KG
(e)

AT ULTIMATE
(f)

CRACKING PATTERN FRI

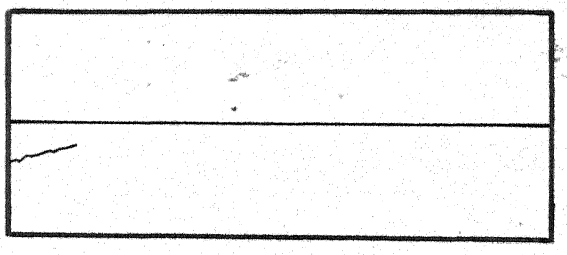


AT LOAD=910 KG
(g)

AT ULTIMATE
(h)

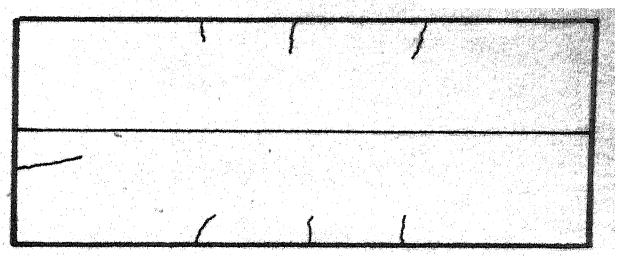
CRACKING PATTERN FR2

FIG.3·6·1 CRACKING PATTERN OF REINFORCED CONCRETE SPECIMENS



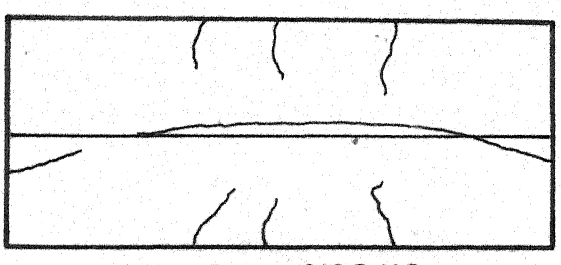
AT LOAD= 2280 KG

(j)



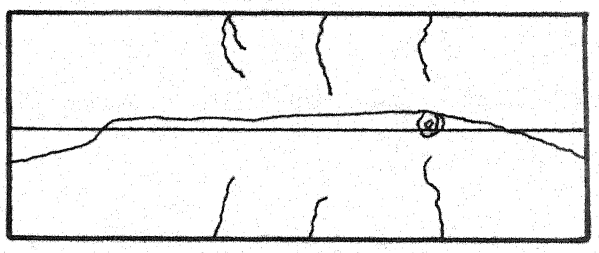
AT LOAD=2710KG

(k)



AT LOAD=3180 KG

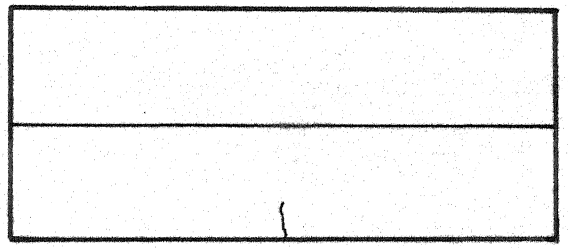
(l)



AT ULTIMATE

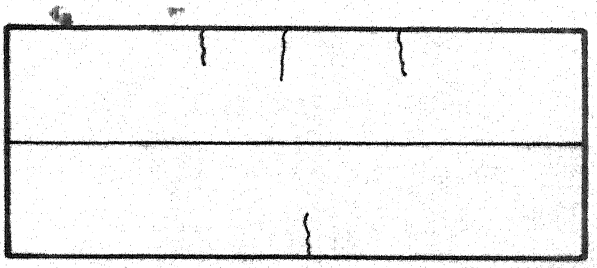
(m)

CRACKING PATTERN OF FP2



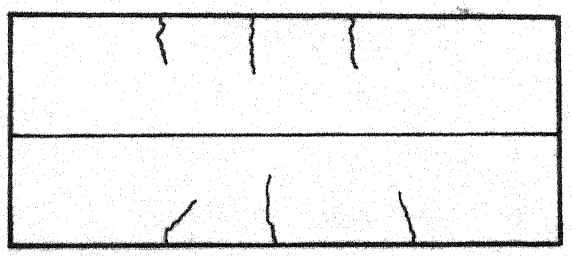
AT LOAD=2280 KG

(n)



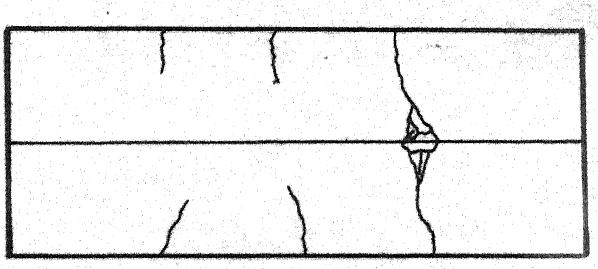
AT LOAD=2710 KG

(p)



AT LOAD=3180 KG

(q)



AT ULTIMATE

(r)

CRACKING PATTERN OF FP3

FIG.3·6·2 CRACKING PATTERN OF PRESTRESSED CONCRETE SPECIMENS

4. CONCLUSIONS

It is observed that the transverse ties have considerable effect on the stiffness of reinforced concrete folded plates. The deflections are decreased by providing the ties to an appriciable extent. The spacing of ties also had considerable effect. The deflections are reduced by about 50% in the case of reinforced specimens with single tie, and with increase in the number of ties to two the deflections are reduced by about 60%. There is probably some relation between the width of the folded plate and the tie spacing such that the effect of tie spacing on deformation is reduced. Table 4.1 gives the effect of tie spacing on the deflections and also its relation with width of the folded plate. Fig. 4.1 indicates the variation of the deformation with spacing-folded width ratio. As the spacing-fold width ratio decreases the deflections decrease less rapidly and becomes asymtotic. It appears that the spacing-fold width ratio of 1 to 1.25 seems to be a limit at which the effect of spacing of the ties reaches a steady state.

In the case of prestressed concrete folded plates also it is observed that the transverse ties have some effect on the stiffness of the specimens. The deflections are decreased by about 25% in the case of specimen with one tie and with the increase in the number of ties to two

the deflection are decreased by about 30%. This shows that the spacing of tie bars has effect on the deformations of the specimens. Table 4.2 gives the effect of tie spacing on deflections and also its relation with width of the fold for the prestressed concrete folded plates. Fig. 4.2 indicates the variation of deformations with spacing-fold width ratio of the prestressed concrete folded plates. Here also as the spacing-fold width ratio decreases the deflections decrease till the spacing-fold width ratio reaches one.

Therefore the recommended spacing in general may be in the range of 1 to 1.25 times the width of the folded plate for both R.C.C. and prestressed concrete folded plates. Schwaighofer and Seethaler[2] made an investigation with two different tie bar spacings and suggested the tie bar spacing and fold width ratio as 1.33.

TABLE 4.1: EFFECT OF SPACING OF TIES ON THE DEFLECTIONS OF R.C.C. FOLDED PLATES.

No.of ties	Ratio of the spacing of the ties to the fold width	Deflection ratio as compared to the deflection of folded plate without ties.
0	3.0	1.0
1	1.5	0.4 to 0.45
2	1.0	0.35 to 0.40

TABLE 4.2: EFFECT OF SPACING OF TIES ON THE DEFLECTIONS OF PRESTRESSED CONCRETE FOLDED PLATES.

No.of ties	Ratio of the spacing of the ties to the fold width.	Deflection ratio as compared to the deflection of folded plate without ties.
0	3.0	1.0
1	1.5	0.7 to 0.75
2	1.0	0.65 to 0.70
3	0.75	0.75 to 0.80

The ultimate strength of the folded plates is not effected by the ties. The reinforced concrete folded plates failed in tension whereas the prestressed concrete folded plates failed by compression. In both the types of failure, the spacing of the ties showed no effect.

The prestressing of the transverse ties have no effect either on stiffness or on the ultimate strength. However the amount of tie prestressing produces considerable secondary bending moments on the transverse and longitudinal plates. Therefore only a very nominal prestressing of the ties is recommended.

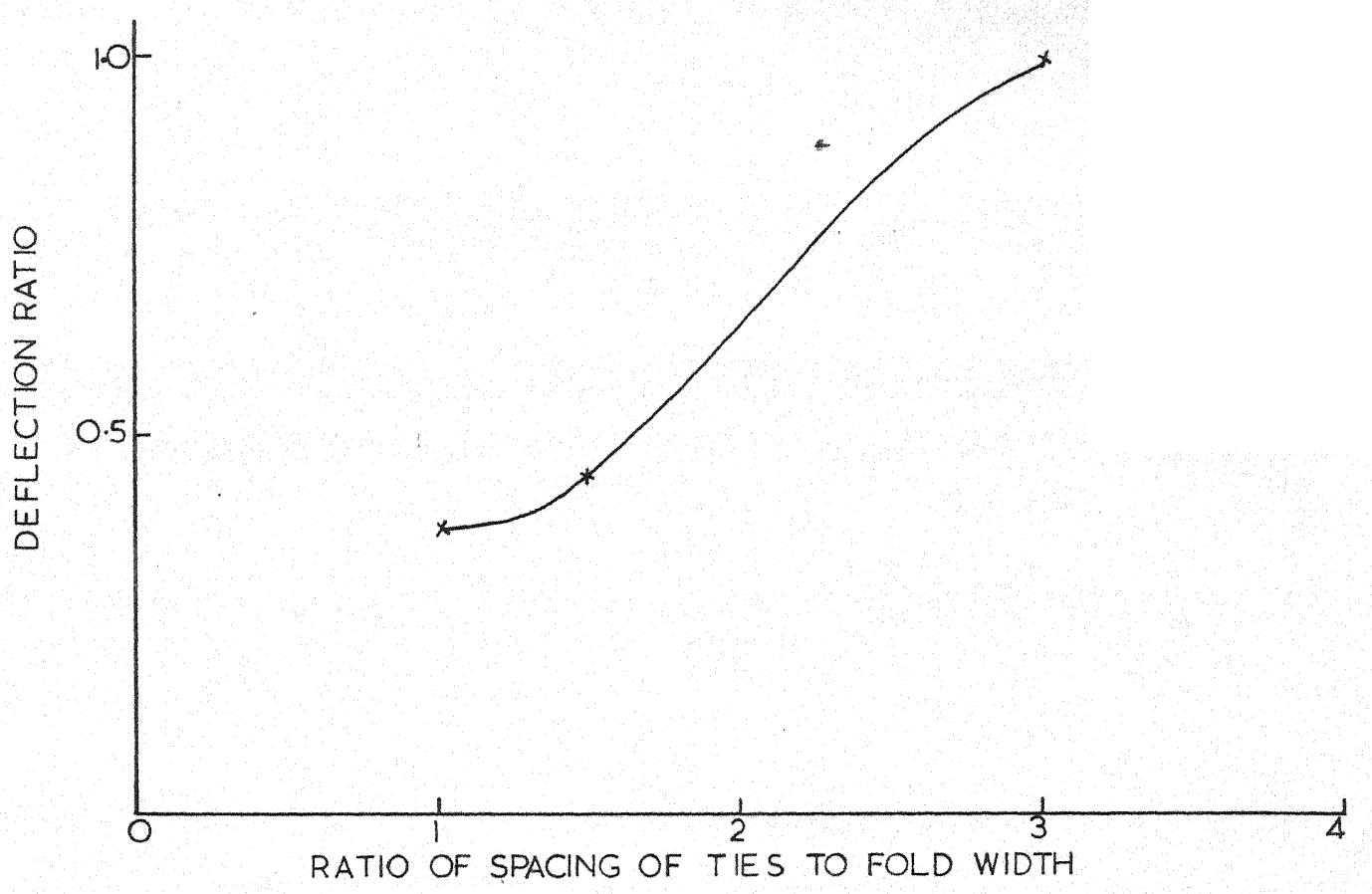


FIG. 4·1 EFFECT OF SPACING OF TIES ON R.C.C. SPECIMENS

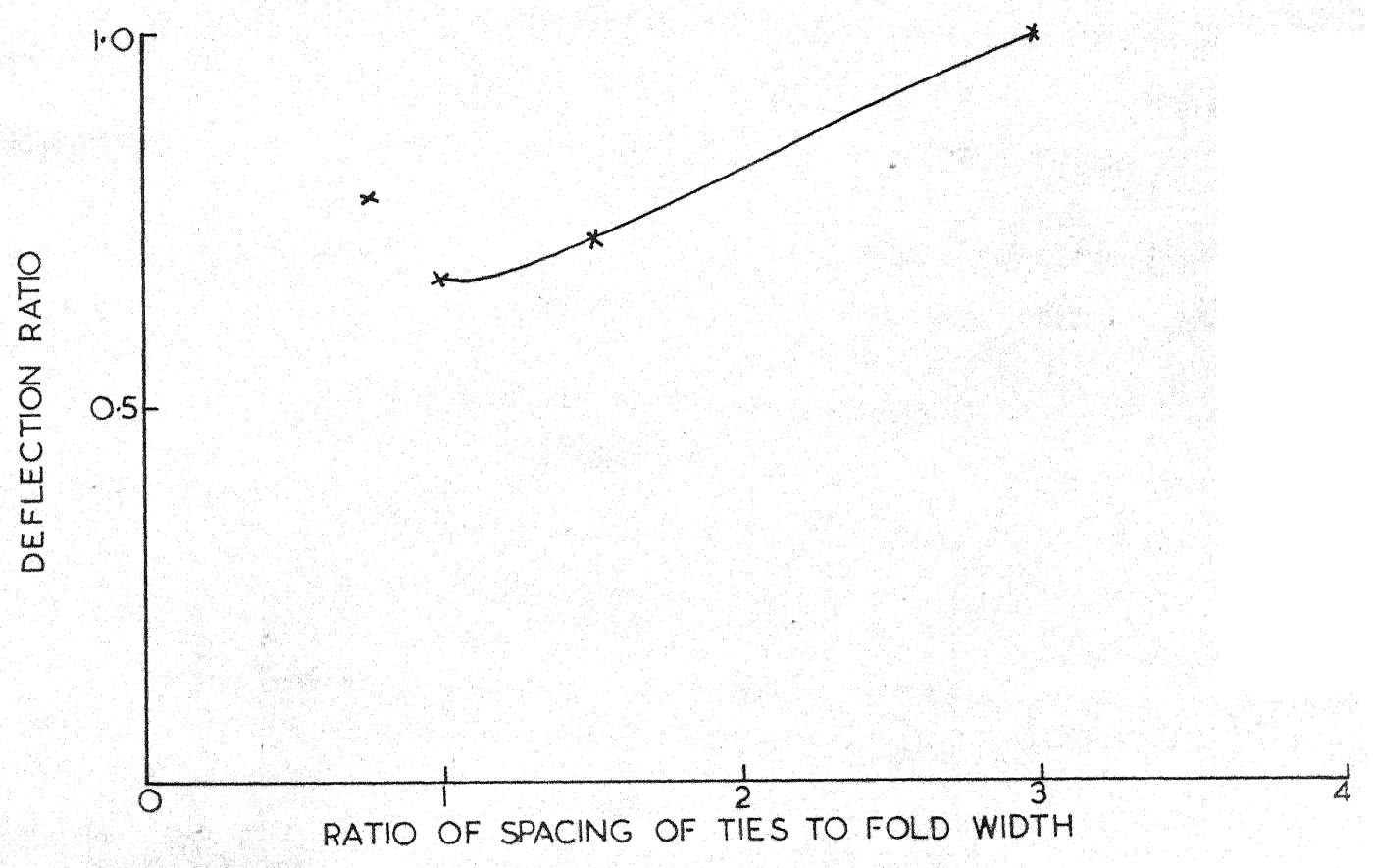


FIG. 4·2 EFFECT OF SPACING OF TIES ON PRESTRESSED CONCRETE SPECIMENS

REFERENCES

(1) Glanville, J.I., "Full-scale pretensioned folded plates test - loaded to failure", Proc. Amer. Concrete Institute, March 1963, pp. 355-370.

(2) Schwaighofer, J. and Seethaler, H., "Experimental study of folded plates", Proc. ACI., Jan. 1963, pp. 101-112.

(3) Ali, A.A., "The analysis of continuous hipped plate structures", Sc.D. Thesis, University of Michigan, 1951, Dissertation Abstracts, University Microfilms Inc., Ann Arbour, p.169, Vol. 12, No. 2, Publ. 3460, 1952.

(4) Winter, G. and Pei, M., "Hipped plate construction", Journ. ACI., Vol. 43, Jan. 1947, pp.505-532.

(5) Craemer, H., "Prismatic structures with transverse stiffeners", Concrete and Constructional Engineering, Vol. 45, No. 3, March 1950, pp. 81-86.

(6) Huang, Pao Chyuan, "The analysis of hipped-plate structures with intermediate diaphragms", Sc. D. Thesis, University of Michigan, 1950, Dissertation Abstracts, Vol. XI, No. 2, pp. 268-269, Publication No. 2413.

(7) Edwards, M.H., "Precast and prestressed folded plate slabs", Proc. of ACI., Vol. 32, April 1961, pp. 1313-1322.

(8) Chacos, G.P., and Scalzi, J.B., "Ultimate strength of a folded plate structure", Proc. ACI, Vol. 32, Feb. 1961, pp. 965-971.

(9) Aldridge, W.W., "Ultimate strength tests of model reinforced concrete folded plate structures", Ph.D. Thesis, University of Texas, 1966, Dissertation abstracts, University Microfilms, Inc., Ann Arbor, Michigan.

APPENDIX - ULTIMATE STRENGTH CALCULATIONS

ULTIMATE STRENGTH CALCULATIONS

Reinforced Folded Plates:

The ultimate strength of the folded plates was calculated using equivalent rectangular theory.

Area $A = 2 \times 2.5 \times 30 = 150$ sq.cm.

Equivalent rectangle width $b = \frac{150}{15} = 10$ cm.

Ultimate strength of FP0:

$f'_c = 470$ kg./cm^2

$f_{su} = 7900$ kg/cm^2

$f_y = 6950$ kg/cm^2

Tension steel ration $p = \frac{8 \times 0.079}{150} = 0.00421$

Compression steel ratio $p' = \frac{1 \times 0.079}{150} = 0.00053$

$p - p' = 0.00368$

$$p_{max} = 0.75 \times 0.85 \times 0.75 \times \frac{470}{6950} \times \frac{87000}{87000 + 10000}$$

$$= 0.0151$$

As $(p-p')$ is less than p_{max} neglect compression reinforcement.

$d = \frac{19.05}{2} = 9.525$

$$M_u = P_{su}\left(d - \frac{0.6\ P_{su}}{b\ f'_c}\right)$$

$$= 8 \times 630\ \left(9.525 - \frac{0.6 \times 8 \times 630}{10 \times 470}\right) = 44,700\ kg\text{-}cm$$

$$P_u = \frac{44700 \times 3 \times 2}{154} = 1740\ kg.$$

Ultimate Strength of PR1:

$$f'_c = 504\ kg/cm^2$$

$$p = \frac{6 \times 0.079}{150} = 0.00316$$

$$p' = \frac{0.079}{150} = 0.00053$$

$$p - p' = 0.00263$$

Here also as (p-p') is less than p_{max} neglect compression reinforcement.

$$d = \frac{15.24}{2} = 7.62\ cm.$$

$$M_u = 6 \times 630\ (7.62 - \frac{0.6 \times 6 \times 630}{10 \times 504}) = 27200\ kg\text{-}cm.$$

$$P_u = \frac{27200 \times 3 \times 2}{154} = 1,060\ kg.$$

Ultimate strength of PR2:

$$f'_c = 525\ kg/cm^2$$

$$p = 0.00316$$

$$p' = 0.00053$$

Here also as $(p-p')$ is less than p_{max} neglect compression reinforcement.

$d = 7.62$ cm.

$$M_u = 6 \times 630 \left(7.62 - \frac{0.6 \times 6 \times 630}{10 \times 525}\right) = 27,200 \text{ kg-cm.}$$

$P_u = 1,060$ kg.

Ultimate strength of prestressed folded plates:

Using equivalent rectangle theory

Area $= 2 \times 2.5 \times 30 = 150$ sq.cm.

Equivalent rectangle width $b = \frac{150}{15} = 10$ cm.

Ultimate strength of PPO:

$$f'_c = 470 \text{ kg/cm}^2$$

$$f_{su} = 17625 \text{ kg/cm}^2$$

$$\frac{p_{fsu}}{f'_c} = \frac{4 \times 0.196}{150} \times \frac{17625}{470} = 0.206$$

As $\frac{p_{fsu}}{f'_c}$ is less than 0.3 use

$M_u = A_s f_{su} (d - a/2)$ where $a = \frac{A_s f_{su}}{0.85 f'_c b}$

$$a = \frac{4 \times 0.196 \times 17625}{0.85 \times 470 \times 10} = 3.47 \text{ cm.}$$

$d = (20/2) = 10$ cm.

$$M_u = 4 \times 0.196 \times 17625 \left(10 - \frac{3.47}{2}\right) = 114400 \text{ kg-cm.}$$

$$P_u = \frac{114400 \times 3 \times 2}{154} = 4460 \text{ kg.}$$

Ultimate strength of PP1:

$$f'_c = 500 \text{ kg/cm}^2$$

$$\frac{p f_{su}}{f'_c} = \frac{4 \times 0.196}{150} \times \frac{17625}{500} = 0.184 < 0.3$$

$$\therefore M_u = A_s f_{su} (d - a/2)$$

$$a = \frac{4 \times 0.196}{0.85 \times 10} \times \frac{17625}{500} = 3.24 \text{ cm.}$$

$$M_u = 4 \times 0.196 \times 17625 \left(10 - \frac{3.24}{2}\right) = 115800 \text{ kg-cm.}$$

$$P_u = \frac{115800 \times 3 \times 2}{154} = 4500 \text{ kg.}$$

Ultimate strength of PP2:

$$f'_c = 530 \text{ kg/cm}^2$$

Here also as $\frac{p f_{su}}{f'_c} < 0.3$ use

$$M_u = A_s f_{su} (d - a/2)$$

$$a = \frac{4 \times 0.196}{0.85 \times 10} \times \frac{17625}{530} = 3.07 \text{ cms.}$$

$$M_u = 4 \times 0.196 \times 17625 \left(10 - \frac{3.07}{2}\right) = 117000 \text{ kg-cm.}$$

$$P_u = \frac{117000 \times 3 \times 2}{154} = 4560 \text{ kg.}$$

Ultimate strength of PP3:

$f'_c = 495 \ kg/cm^2$

Here also $\frac{p f_{su}}{f'_c} < 0.3$

$\therefore M_u = A_s \ f_{su} \ (d - a/2)$

$a = \frac{4 \times 0.196}{0.85 \times 10} \times \frac{17625}{495} = 3.29$ cms.

$M_u = 4 \times 0.196 \times 17625 \ (10 - \frac{3.29}{2}) = 115400$ kg-cm.

$P_u = \frac{115400 \times 3 \times 2}{154} = 4496$ kg.

Ultimate strength of PP4:

$f'_c = 444 \ kg/cm^2$

$\frac{p f_{su}}{f'_c} = \frac{4 \times 0.196}{150} \quad \frac{17625}{444} = 0.214 < 0.3$

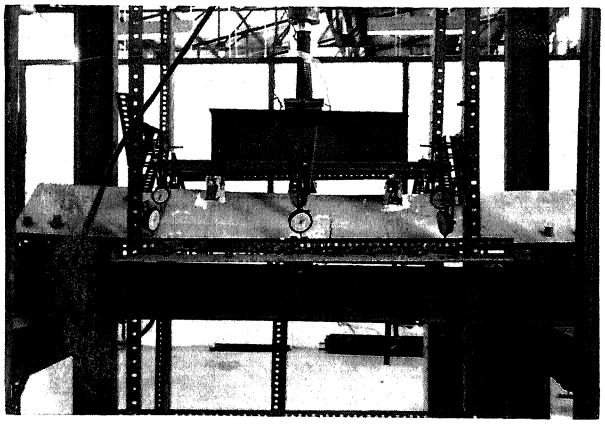
$\therefore M_u = A_s \ f_{su} \ (d - a/2)$

$a = \frac{4 \times 0.196}{0.85 \times 10} \times \frac{17625}{444} = 3.78$

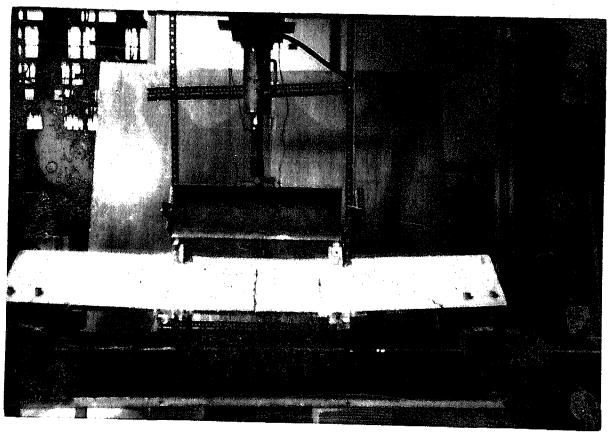
$M_u = 4 \times 0.196 \times 17625 \ (10 - \frac{3.78}{2}) = 112200$ kg-cm.

$P_u = \frac{112200 \times 3 \times 2}{154} = 4380$ kg.

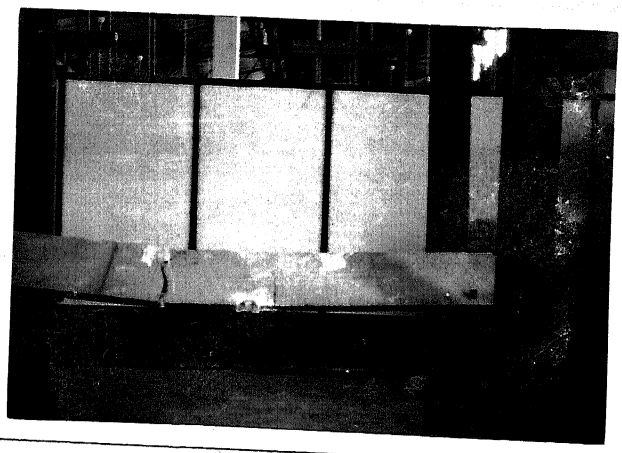
PHOTOGRAPHS



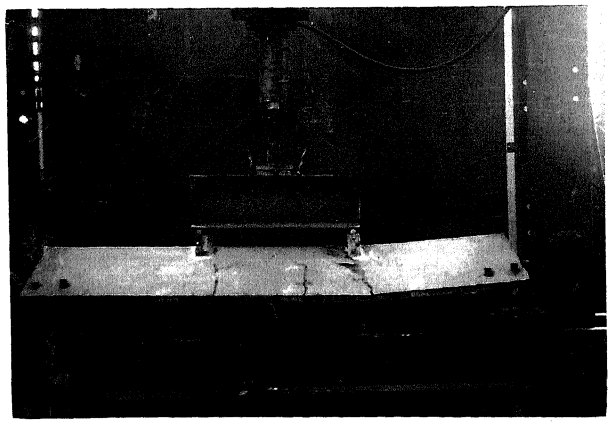
Dial Gauge Arrangement



Loading Arrangement



Folded Plate After Failure



Folded Plate at Failure



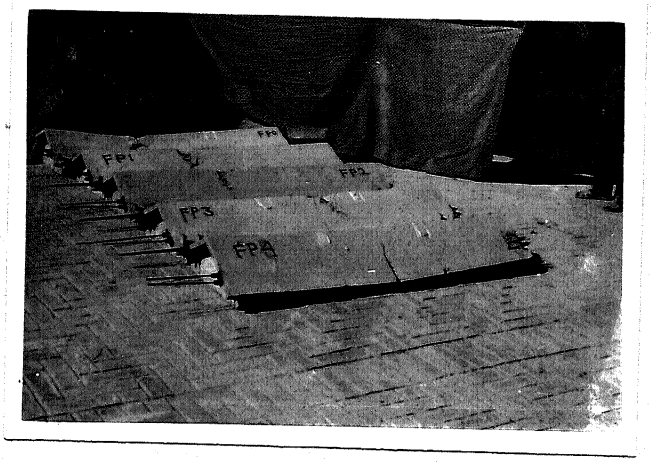
Cracking Pattern of FR2 After Failure



Cracking Patter of FR2 After Failure



Cracking Pattern of FP4 After Failure



Cracking Pattern of Prestressed Concrete Specimens After Failure